3.7 Seismic Design

Plant structures, systems, and components important to safety are required by General Design Criterion (GDC) 2 of Appendix A of 10 CFR 50 to be designed to withstand the effects of earthquakes without loss of capability to perform their safety functions.

Each plant structure, system, equipment, and component is classified in an applicable seismic category depending on its function. A three-level seismic classification system is used for the AP1000: seismic Category I, seismic Category II, and nonseismic. The definitions of the seismic classifications and a seismic classifications listing of structures, systems, equipment, and components are presented in Section 3.2.

Seismic design of the AP1000 seismic Categories I and II structures, systems, equipment, and components is based on the safe shutdown earthquake (SSE). The safe shutdown earthquake is defined as the maximum potential vibratory ground motion at the generic plant site as identified in Section 2.5.

The operating basis earthquake (OBE) has been eliminated as a design requirement for the AP1000. Low-level seismic effects are included in the design of certain equipment potentially sensitive to a number of such events based on a percentage of the responses calculated for the safe shutdown earthquake. Criteria for evaluating the need to shut down the plant following an earthquake are established using the cumulative absolute velocity approach according to EPRI Report NP-5930 (Reference 1) and EPRI Report TR-100082 (Reference 17). For the purposes of the shutdown criteria in Reference 1 the operating basis earthquake for shutdown is considered to be one-third of the safe shutdown earthquake.

Seismic Category I structures, systems, and components are designed to withstand the effects of the safe shutdown earthquake event and to maintain the specified design functions. Seismic Category II and nonseismic structures are designed or physically arranged (or both) so that the safe shutdown earthquake could not cause unacceptable structural interaction with or failure of seismic Category I structures, systems, and components.

3.7.1 Seismic Input

The geologic and seismologic considerations of the plant site are discussed in Section 2.5.

The peak ground acceleration of the safe shutdown earthquake, now referred to as the Certified Seismic Design Response Spectra (CSDRS), has been established as 0.30g for the AP1000 design. The vertical peak ground acceleration is conservatively assumed to equal the horizontal value of 0.30g as discussed in Section 2.5.

3.7.1.1 Design Response Spectra

The AP1000 design response spectra of the safe shutdown earthquake, now referred to as the Certified Seismic Design Response Spectra (CSDRS), are provided in Figures 3.7.1-1 and 3.7.1-2 for the horizontal and the vertical components, respectively.

The horizontal design response spectra for the AP1000 plant are developed, using the Regulatory Guide 1.60 spectra as the base and several evaluations to investigate the high frequency amplification effects. These evaluations included:

- Comparison of Regulatory Guide 1.60 spectra with the spectra predicted by recent eastern U.S. spectral velocity attenuation relations (References 23, 24, 25, and 26) using a suite of magnitudes and distances giving a 0.3 g peak acceleration

- Comparison of Regulatory Guide 1.60 spectra with the 10^{-4} annual probability uniform hazard spectra developed for eastern U.S. nuclear power plants by both Lawrence Livermore National Laboratory (Reference 27) and Electric Power Research Institute (Reference 28)

- Comparison of Regulatory Guide 1.60 spectra with the spectra of 79 additional old and newer components of strong earthquake time histories not considered in the original derivation of Regulatory Guide 1.60

Based on the above described evaluations, it is concluded that the eastern U.S. seismic data exceed Regulatory Guide 1.60 spectra by a modest amount in the 15 to 33 hertz frequency range when derived either from published attenuation relations or from the 10^{-4} annual probability of exceedance uniform hazard spectra at eastern U.S. sites. This conclusion is consistent with findings of other investigators that eastern North American earthquakes have more energy at high frequencies than western earthquakes. Exceedance of Regulatory Guide 1.60 spectra at the high frequency range, therefore, would be expected since Regulatory Guide 1.60 spectra are based primarily on western U.S. earthquakes. The evaluation shows that, at 25 hertz (approximately in the middle of the range of high frequencies being considered, and a frequency for which spectral amplitudes are explicitly evaluated) the mean-plus-one-standard-deviation spectral amplitudes for 5 percent damping range from about 2.1 to 4 cm/sec and average 2.7 cm/sec. Whereas, the Regulatory Guide 1.60 spectral amplitude at the same frequency and damping value equal just over 2 cm/sec.

It is concluded, therefore, that an appropriate augmented 5 percent damping horizontal design velocity response spectrum for the AP1000 project is one with spectral amplitudes equal to the Regulatory Guide 1.60 spectrum at control frequencies 0.25, 2.5, 9 and 33 hertz augmented by an additional control frequency at 25 hertz with an amplitude equal to 3 cm/sec. This spectral amplitude equals 1.3 times the Regulatory Guide 1.60 amplitude at the same frequency. The additional control point's spectral amplitude of other damping values were determined by increasing the Regulatory Guide 1.60 spectral amplitude by 30 percent.

The AP1000 design vertical response spectrum is, similarly, based on the Regulatory Guide 1.60 vertical spectra at lower frequencies but is augmented at the higher frequencies equal to the horizontal response spectrum.

The AP1000 design response spectra's relative values of spectrum amplification factors for control points are presented in Table 3.7.1-3.

The design response spectra are applied at the foundation level in the free field at hard rock sites and at the finished grade in the free field at firm rock and soil sites. The resulting peak horizontal ground acceleration values are above 0.1g. This satisfies 10 CFR Part 50, Appendix S, which requires that the horizontal component of the SSE ground motion in the free-field at the foundation elevation (that is, bottom of foundation) has a peak ground acceleration of at least 0.1g together with an appropriate response spectrum. The definitions (characteristics) of hard rock, firm rock, and soil sites are provided in Subsection 3.7.1.4.

3.7.1.1.1 Soil Profiles and Input Motions for Soil-Structure Interaction Analysis

The site-specific Ground Motion Response Spectra (GMRS) are described in Subsection 2.5.2. The development of the Foundation Input Response Spectra (FIRS) is described in Subsection 3.7.1.1.1.1. The development of Safe Shutdown Earthquake (SSE) motion is provided in Subsection 3.7.1.1.1.2. Strain-compatible soil properties are presented in Subsection 3.7.1.1.1.3. The development of acceleration time histories for soil-structure interaction (SSI) analysis is summarized in Subsection 3.7.1.1.1.4. A detailed discussion of each of these steps is provided in Appendix 3JJ, including a description of the sensitivity assessment in Subsection 3JJ.7 based on updated geotechnical properties provided in Subsection 2.5.4.

The developed input is used for SSI analysis, which is provided in Appendix 3KK. In addition, Appendix 3KK includes a sensitivity assessment based on information described in Subsection 3JJ.7 which considers the updated geotechnical properties presented in Subsection 2.5.4.

3.7.1.1.1.1 Development of FIRS

The Uniform Hazard Response Spectra (UHRS), described in Subsection 2.5.2.4, are defined for hard rock characterized with a shear wave velocity of Vs = 9200 feet/second (2.8 kilometers/second), which is located at about 10,000 feet (3000 meters) below the ground surface. Subsection 2.5.2.5 describes the development of the site amplification factors at the GMRS horizon. Subsection 2.5.2.6 discusses the development of the horizontal and vertical GMRS. The same procedures are followed in this section to develop FIRS at the bottom of the nuclear island foundation horizon.

The full soil columns used for computation of soil amplification factors represent the two soil conditions found at the location of Units 6 & 7. The soil column far from the nuclear island consists of in situ soil layers except for the upper 30.5 feet (9.3 meters) of structural fill. This is the fill required for the general plant area to raise the site grade elevation from the existing grade to the finished grade, and is designated as "FAR" in this Section. A second soil column, representing the site conditions near the nuclear island, where, in addition to the general fill, lean concrete, and structural fill replace the in situ soils down to a depth of 60.5 feet (18.4 meters) below finished grade. This is designated "NI" in the following discussion.

The site response analysis is conducted on a set of 60 randomized profiles, for each of the two soil profiles, to account for the variability in the dynamic soil properties. The randomization procedure is described in detail in Subsection 2.5.2.5.2. Using the randomized soil profiles, the soil column analyses are performed with the de-aggregated low frequency (LF) and high frequency (HF) spectra of the hard rock motion at 10^{-4} and 10^{-5} annual-frequency-of-exceedance, presented in Subsection 2.5.2.4, following the same methodology described in Subsection 2.5.2.5. Log-mean amplification functions and soil response spectra are developed for "outcrop" motions for both FAR and NI soil conditions at the FIRS horizon, located at the bottom of the nuclear island foundation at elevation -16 feet (-4.9 meters) corresponding to a depth of 41.5 feet (12.7 meters) below finished ground surface.

FIRS are computed at the elevation of -16 feet from the envelope of NI and FAR soil columns representing near and far field soil columns. To satisfy the requirements of Appendix S to 10 CFR Part 50, for the purpose of SSI analysis, the horizontal and vertical SSE motions are calculated as the envelope of FIRS and the Regulatory Guide 1.60 motions scaled to 0.1 g peak ground acceleration (PGA). The input for SSI analysis in terms of acceleration time histories were computed as in-column motion corresponding to the SSE as described in Section 3JJ.6 in Appendix 3JJ. In application of the SSI input acceleration time histories, the control point was defined at elevation of -14 feet at the bottom of the NI basemat.

The change of 2 feet accounts for the thickness of the mud mat(s) and the water proofing membrane. This is considered acceptable since, in computation of the FIRS for NI soil column, 19 feet of lean concrete is already included in the soil column analysis and an additional 2 feet of concrete has negligible effects on the FIRS and associated SSI input motion.

Following the same procedure as used in Subsection 2.5.2.6 to obtain the GMRS, the procedure presented in Regulatory Guide 1.208 is implemented to develop the horizontal design response spectrum (DRS) for each of the FAR and NI soil columns. The horizontal FIRS is defined as the envelope of the FAR and NI DRS. The vertical FIRS is obtained by scaling the horizontal FIRS by the same V/H as presented in Subsection 2.5.2.6. The details of the site response analysis and development of FIRS are documented in Appendix 3JJ.

In addition to the FIRS, from the same set of soil amplification analyses, design spectra at the ground surface for both NI and FAR soil profiles are developed and enveloped. The surface DRS are used to check the adequacy of the SSI input motion as described in Subsection 3.7.1.1.1.4 and Appendix 3JJ.

The resulting horizontal and vertical FIRS are plotted in Figure 3.7-201 and reported in Table 3.7-201. As developed and described in Appendix 3KK, comparisons of the FIRS developed indicate they are enveloped completely by the AP1000 Certified Seismic Design Response Spectra (CSDRS) for all frequencies, as shown in Figure 3.7-202. The analysis results show that the Nuclear Island Floor Response Spectra (FRS) of AP1000 at the Turkey Point site at six key locations are enveloped by the AP1000 Certified Design Response Spectra (CSDRS).

3.7.1.1.1.2 Safe Shutdown Earthquake Motion

To satisfy the requirements of Appendix S to 10 CFR Part 50, namely that the SSE motion, for the purpose of soil-structure interaction analysis, must be an adequate acceleration response spectra (ARS) with a minimum PGA of 0.1 g (also referred to as the minimum required response spectra), the site-specific FIRS and the minimum required response spectra are enveloped. The resulting 5 percent damping ARS (horizontal and vertical) are considered the SSE motions for the site. More details on the comparison of the FIRS with the minimum required response spectra and the calculation of the horizontal and vertical SSE ARS are provided in Appendix 3JJ.

3.7.1.1.1.3 Strain-Compatible Soil Property Profiles

Two sets of soil profile properties corresponding to the FAR and NI site conditions are developed that are strain-compatible with the developed SSE motion. Each set consists of the best estimate (BE), the lower bound (LB) and the upper bound (UB) strain-compatible shear-wave velocity, P-wave velocity and damping profiles. The development of strain-compatible soil profiles is discussed in detail in Appendix 3JJ.

3.7.1.1.1.4 Acceleration Time Histories for SSI Input

Acceleration time histories for use in SSI analysis of the nuclear island (which includes modeling of the embedment of the nuclear island) are presented in this section. The seed acceleration time histories were selected from the database of candidate time histories given in NUREG/CR-6728 based on the low frequency de-aggregation results (i.e., magnitudes > 7 and distances > 500 km). For the analysis, the three component (i.e., two horizontal and one vertical component) strong ground motion recordings from the 1999 Chi-Chi earthquake (magnitude=7.6) recorded at the TAP024 station (closest distance=100.2 km) were selected and matched to the 5 percent damping SSE ARS developed earlier (see Subsection 3.7.1.1.1.2). These time histories were modified to be spectrum-compatible to the SSE ARS following the spectral matching criteria given in NUREG/CR-6728. The acceleration response spectra of the generated time histories matching SSE ARS are shown in Appendix 3JJ.

For SSI input motion of nuclear island with embedment, these acceleration time histories are propagated through the strain-compatible soil profiles, presented in Subsection 3.7.1.1.1.3, where they are used as input “outcrop” motions in the soil column at the FIRS horizon and the “within” acceleration time histories at the same horizon are computed. No further iterations on soil properties are performed. This analysis results in a set of 3 “within” motions for each soil profile in the two horizontal directions (H1 and H2) and vertical direction (UP), respectively. Six (6) sets are developed corresponding to the LB, BE and UB profiles for NI and FAR soil conditions. The analysis also incorporates the requirement for checking the adequacy of the SSI input motion (References 201 and 202). Checks are made with respect to the corresponding surface design response spectra (DRS)

and modifications are made where necessary. The analysis steps are discussed in detail in Appendix 3JJ.

The “within” acceleration time histories are recommended for use in the SSI analysis of the nuclear island SSI model that includes embedment. The time histories are to be applied at the FIRS horizon as “within” motion and shall be used in combination with the respective SSI soil profiles discussed in Subsection 3.7.1.1.1.3.

3.7.1.2 Design Time History

A "single" set of three mutually orthogonal, statistically independent, synthetic acceleration time histories is used as the input in the dynamic analysis of seismic Category I structures. The synthetic time histories were generated by modifying a set of actual recorded "TAFT" earthquake time histories. The design time histories include a total time duration equal to 20 seconds and a corresponding stationary phase, strong motion duration greater than 6 seconds. The acceleration, velocity, and displacement time-history plots for the three orthogonal earthquake components, "H1," "H2," and "V," are presented in Figures 3.7.1-3, 3.7.1-4, and 3.7.1-5. Design horizontal time history, H1, is applied in the north-south (Global X or 1) direction; design horizontal time history, H2, is applied in the east-west (global Y or 2) direction; and design vertical time history is applied in the vertical (global Z or 3) direction. The cross-correlation coefficients between the three components of the design time histories are as follows:

$$\rho_{12} = 0.05,\ \rho_{23} = 0.043,\ \text{and}\ \rho_{31} = 0.140$$

where 1, 2, 3 are the three global directions.

Since the three coefficients are less than 0.16 as recommended in Reference 30, which was referenced by NRC Regulatory Guide 1.92, Revision 1, it is concluded that these three components are statistically independent. The design time histories are applied at the foundation level in the free field.

The ground motion time histories (H1, H2, and V) are generated with time step size of 0.010 second for applications in soil structure interaction analyses. For applications in the fixed-base mode superposition time-history analyses, the time step size is reduced to 0.005 second by linear interpolation. The maximum frequency of interest in the horizontal and vertical seismic analysis of the nuclear island is 33 hertz. Modes with higher frequencies are included in the analysis so that the mass in these higher modes is included in the member forces. The cutoff frequencies used in the soil structure interaction analyses are 33 hertz. The maximum "cut-off" frequency for the soil structure interaction analyses and the fixed-base analyses is well within the Nyquist frequency limit.

The comparison plots of the acceleration response spectra of the time histories versus the design response spectra for 2, 3, 4, 5, and 7 percent critical damping are shown in Figures 3.7.1-6, 3.7.1-7, and 3.7.1-8. The SRP 3.7.1, Table 3.7.1-1, provision of frequency intervals is used in the computation of these response spectra.

In SRP 3.7.1 the NRC introduced the requirement of minimum power spectral density to prevent the design ground acceleration time histories from having a deficiency of power over any frequency range. SRP 3.7.1, Revision 2, specifies that the use of a single time history is justified by satisfying a target power spectral density (PSD) requirement in addition to the design response spectra enveloping requirements. Furthermore, it specifies that when spectra other than Regulatory Guide 1.60 spectra are used, a compatible power spectral density shall be developed using procedures outlined in NUREG/CR-5347 (Reference 29).

The NUREG/CR-5347 procedures involve ad hoc hybridization of two earlier power spectral density envelopes. Since the modification to the RG 1.60 design spectra adopted for AP1000 (see Subsection 3.7.1.1) is relatively small (compared to the uncertainty in the fit to RG 1.60 of power spectral density-compatible time histories referenced in NUREG/CR-5347) and occurs only in the frequency range between 9 to 33 hertz, a project-specific power spectral density is developed using a slightly different hybridization for the higher frequencies.

Since the original RG 1.60 spectrum and the project-specific modified RG 1.60 spectrum are identical for frequencies less than 9 hertz, no modification to the power spectral density is done in this frequency range. At frequencies above 9 hertz, the third and the fourth legs of the power spectral density are slightly modified as follows:

- The frequency at which the design response spectrum inflected towards a 1.0 amplification factor at 33 hertz takes place at 25 hertz in the AP1000 spectrum rather than at 9 hertz as in the RG 1.60 spectrum. The third leg of the power spectral density, therefore, is extended to about 25 hertz rather than 16 hertz.

- The lead coefficient to the fourth leg of the power spectral density is changed to connect with the extended third leg.

The AP1000 augmented power spectral density, anchored to 0.3 g, is as follows:

$$S_0(f) = 58.5\ (f/2.5)^{0.2}\ in^2/sec^3,\ f \le 2.5\ \text{hertz}$$

$$S_0(f) = 58.5\ (2.5/f)^{1.8}\ in^2/sec^3,\ 2.5\ \text{hertz} \le f \le 9\ \text{hertz}$$

$$S_0(f) = 5.832\ (9/f)^3\ in^2/sec^3,\ 9\ \text{hertz} \le f \le 25\ \text{hertz}$$

$$S_0(f) = 0.27\ (25/f)^8\ in^2/sec^3,\ 25\ \text{hertz} \le f$$

The AP1000 Minimum Power Spectral Density is presented in Figure 3.7.1-9. This AP1000 target power spectral density is compatible with the AP1000 horizontal design response spectra and envelops a target power spectral density compatible with the AP1000 vertical design response spectra. This AP1000 target power spectral density, therefore, is conservatively applied to the vertical response spectra.

The comparison plots of the power spectral density curve of the AP1000 acceleration time histories versus the target power spectral density curve are presented in Figures 3.7.1-10, 3.7.1-11, and 3.7.1-12. The power spectral density functions of the design time histories are calculated at uniform frequency steps of 0.0489 hertz. The power spectral densities presented in Figures 3.7.1-10 through 3.7.1-12 are the averaged power spectral density obtained over a moving frequency band of ±20 percent centered at each frequency. The power spectral density amplitude at frequency (f) has the averaged power spectral density amplitude between the frequency range of 0.8 f and 1.2 f as stated in appendix A of Revision 2 of SRP 3.7.1.

3.7.1.3 Critical Damping Values

Energy dissipation within a structural system is represented by equivalent viscous dampers in the mathematical model. The damping coefficients used are based on the material, load conditions, and type of construction used in the structural system. The safe shutdown earthquake damping values used in the dynamic analysis of various structures, supports, and equipment are presented in Table 3.7.1-1. The damping values are based on Regulatory Guide 1.61, Revision 0, ASCE Standard 4-98 (Reference 3), except for the damping value of the primary coolant loop piping, which is based on Reference 22, and conduits, cable trays and their related supports.

The damping values for conduits, cable trays and their related supports are shown in Table 3.7.1-1. The damping value of conduit, empty cable trays, and their related supports is similar to that of a bolted structure, namely 7 percent of critical. The damping value of filled cable trays and supports increases with increased cable fill and level of seismic excitation. Full cable trays use a 10-percent damping value consistent with RG 1.61, Revision 1. The limiting condition for design of the AP1000 standard cable tray supports is for full cable tray weight.

For structures or components composed of different material types, the composite modal damping is calculated using the stiffness-weighted method based on Reference 3. The modal damping values equal:

$$\beta_n = \sum_{i=1}^{nc} \frac{\{\phi_n\}^T \beta_i \, [K_t]_i \, \{\phi_n\}}{\{\phi_n\}^T \, [K_t] \, \{\phi_n\}}$$

where:

β_n	=	ratio of critical damping for mode n
nc	=	number of elements
$\{\phi_n\}$	=	mode n (eigenvector)
$[K_t]_i$	=	stiffness matrix of element i
β_i	=	ratio of critical damping associated with element i
$[K_t]$	=	total system stiffness matrix

The linear structural damping values were defined in the modeling codes as a parameter of material property defined for each element. This form of structural damping is used for seismic time history analyses. The structural models analyzed follow the damping criteria stated in Table 3.7.1-1 using 5-percent SSE damping for steel composite (SC) structures, including the shield building wall and modules, and 7-percent SSE damping for the remaining reinforced concrete (RC) structures throughout the nuclear island. A time history non-linear analysis confirms only minor cracking in the nuclear island structure.

3.7.1.4 Supporting Media for Seismic Category I Structures

The supporting media will be described consistent with the information items in Subsection 2.5.4. Seismic analyses for both rock and soil sites are described in Subsection 3.7.2 and Appendix 3G.

The AP1000 nuclear island consists of three seismic Category I structures founded on a common basemat. The three structures that make up the nuclear island are the coupled auxiliary and shield buildings, the steel containment vessel, and the containment internal structures. [*The nuclear island is shown in Figure 3.7.1-14.*]* The foundation embedment depth, foundation size, and total height of the seismic Category I structures are presented in Table 3.7.1-2.

For the design of seismic Category I structures, a set of six design soil profiles (that include hard rock) of various shear wave velocities is established from parametric studies as described in Appendix 3G.3. The soil cases selected for the AP1000 use parameters from the AP600 design, and

*NRC Staff approval is required prior to implementing a change in this information.

the AP600 conclusions are applicable to the AP1000 due to the identical footprint to the AP600 and the similarity in overall mass.

For the AP1000 2D and 3D soil-structure interaction analyses, although some of the parabolic soil profiles are defined using a depth of 240 feet, the actual soil profile defined in SASSI (System for Analysis of Soil-Structure Interaction) (base rock) goes to only elevation 120′.

Soil-structure interaction analyses on soil sites for the AP1000 used the latest soil degradation curves recommended by EPRI TR-102293, although these represent more recent soils data and differ slightly for those used for the AP600.

These six profiles are sufficient to envelope sites where the shear wave velocity of the supporting medium at the foundation level exceeds 1000 feet per second (see Subsection 2.5.2). The design soil profiles include a hard rock site, a soft rock site, a firm rock site, an upper bound soft-to-medium soil site, a soft-to-medium soil site, and a soft soil site. The shear wave velocity profiles and related governing parameters of the six sites considered are as follows:

- For the hard rock site, an upper bound case for rock sites using a shear wave velocity of 8000 feet per second.
- For the firm rock site, a shear wave velocity of 3500 feet per second to a depth of 120 feet and base rock at the depth of 120 feet.
- For the soft rock site, a shear wave velocity of 2400 feet per second at the ground surface, increasing linearly to 3200 feet per second at a depth of 240 feet, and base rock at the depth of 120 feet.
- For the upper bound soft-to-medium soil site, a shear wave velocity of 1414 feet per second at ground surface, increasing parabolically to 3394 feet per second at 240 feet, base rock at the depth of 120 feet, and ground water at grade level. The initial soil shear modulus profile is twice that of the soft-to-medium soil site.
- For the soft-to-medium soil site, a shear wave velocity of 1000 feet per second at ground surface, increasing parabolically to 2400 feet per second at 240 feet, base rock at the depth of 120 feet, and ground water is assumed at grade level.
- For the soft soil site, a shear wave velocity of 1000 feet per second at ground surface, increasing linearly to 1200 feet per second at 240 feet, base rock at the depth of 120 feet, and ground water is assumed at grade level

The strain-dependent shear modulus curves for the foundation materials, together with the corresponding damping curves are taken from References 37 and 38 and are shown in Figures 3.7.1-15 and 3.7.1-16 for rock material and soil material respectively. The different curves for soil in Figure 3.7.1-16 apply to the range of depth within a soil column below grade. The strain-dependent soil material damping is limited to 15 percent of critical damping. The strain-dependent properties used in the SSI analyses for the safe shutdown earthquake are shown in Table 3.7.1-4 and Figure 3.7.1-17 for the firm rock, soft rock, upper bound soft-to-medium soil, soft-to-medium soil, and soft soil properties.

Some variation of soil modeling (water table, soil layering, soil degradation model, and the like) and combinations of these have been demonstrated to have no significant effect on the seismic response of the nuclear island structures. The governing parameters obtained for the AP600 soil studies are also applicable to the AP1000. Each of the parameters deemed not significant has been analyzed.

For instance, the combination of effects of the different strain dependent soil parameters that affect the strain-iterated shear wave velocity profiles was evaluated and shown not to result in exceedances of the envelope of the generic seismic design in-structure response spectra (ISRS).

3.7.2 Seismic System Analysis

Seismic Category I structures, systems, and components are classified according to Regulatory Guide 1.29. Seismic Category I building structures of AP1000 consist of the containment building (the steel containment vessel and the containment internal structures), the shield building, and the auxiliary building. These structures are founded on a common basemat and are collectively known as the nuclear island or nuclear island structures. [*Key dimensions, such as thickness of the basemat, floor slabs, roofs and walls, of the seismic Category I building structures are shown in Figure 3.7.2-12.*]*

Seismic systems are defined, according to SRP 3.7.2, Section II.3.a, as the seismic Category I structures that are considered in conjunction with their foundation and supporting media to form a soil-structure interaction model. The following subsections describe the seismic analyses performed for the nuclear island. Other seismic Category I structures, systems, equipment, and components not designated as seismic systems (that is, heating, ventilation, and air-conditioning systems; electrical cable trays; piping systems) are designated as seismic subsystems. The analysis of seismic subsystems is presented in Subsection 3.7.3.

Seismic Category I building structures are on the nuclear island. Other building structures are classified nonseismic or seismic Category II. Nonseismic structures are analyzed and designed for seismic loads according to the Uniform Building Code (Reference 2) requirements for Zone 2A. Seismic Category II building structures are designed for the safe shutdown earthquake using the same methods and design allowables as are used for seismic Category I structures. The acceptance criteria are based on ACI 349 for concrete structures and on AISC N690 for steel structures including the supplemental requirements described in Subsections 3.8.4.4.1 and 3.8.4.5. The seismic Category II building structures are constructed to the same requirements as the nonseismic building structures, ACI 318 for concrete structures and AISC-S355 for steel structures.

Separate seismic analyses are performed for the nuclear island for each of the six design soil profiles defined in Subsection 3.7.1.4. The analyses generate one set of in-structure responses for each of the design soil profiles. The six sets of in-structure responses are enveloped to obtain the seismic design envelope (design member forces, nodal accelerations, nodal displacements, and floor response spectra), which are used in the design and analysis of seismic Category I structures, components, and seismic subsystems.

Appendix 3G summarizes the types of models and analysis methods that are used in the seismic analyses of the nuclear island, as well as the type of results that are obtained and where they are used in the design. The seismic analyses of the nuclear island are summarized in a seismic analysis summary report. This report describes the development of the finite element models, the soil structure interaction and fixed base analyses, and the results thereof. Seismic response spectra are given in Appendix 3G for the six key locations:

- Containment internal structures at reactor vessel support elevation 100.00′.
- Containment internal structures at operating deck elevation 134.25′.
- Auxiliary shield building north east corner at control room floor elevation 116.50′.
- Auxiliary shield building corner of fuel building roof at shield building elevation 179.19′.
- Auxiliary shield building roof area elevation 327.41′.
- Steel containment vessel near polar crane elevation 224.000′.

*NRC Staff approval is required prior to implementing a change in this information.

3.7.2.1 Seismic Analysis Methods

Seismic analyses of the nuclear island are performed in conformance with the criteria within SRP 3.7.2.

Seismic analyses – using response spectra analysis, the equivalent static acceleration method, the mode superposition time-history method, and the complex frequency response analysis method – are performed for the safe shutdown earthquake to determine the seismic force distribution for use in the design of the nuclear island structures, and to develop in-structure seismic responses (accelerations, displacements, and floor response spectra) for use in the analysis and design of seismic subsystems.

3.7.2.1.1 Equivalent Static Acceleration Analysis

Equivalent static analyses, using computer program ANSYS (Reference 36), are performed to obtain the seismic forces and moments required for the structural design of the steel containment vessel and the nuclear island basemat (see Subsection 3.8.2.4.1.1). Equivalent static loads are applied to the finite element models using the maximum acceleration results from the time history analyses for the six design soil profiles. Accidental torsional moments are applied as described in Subsection 3.7.2.11.

Equivalent static analyses are also performed for design of the shield building roof and radial roof beams, PCS tank, tension ring, and air inlet structure (see Subsection 3.8.4.4.1). The equivalent static loads are based on the maximum acceleration results from time history dynamic analysis of the nuclear island in Subsection 3.7.2.1.2.

3.7.2.1.2 Time-History Analysis and Complex Frequency Response Analysis

Mode superposition time-history analyses using computer program ANSYS and complex frequency response analysis using computer program SASSI are performed to obtain the in-structure seismic response needed in the analysis and design of seismic subsystems. Three-dimensional finite element shell models of the nuclear island structures are used in conjunction with the design soil profiles presented in Subsection 3.7.1.4 to obtain the in-structure responses. Stick models are coupled to the shell models of the concrete structures for the containment vessel, polar crane, reactor coolant loop, pressurizer, and core makeup tanks. Three models are used. The fine (NI10) model, as described in Subsection 3G.2.2.1, is used to define the seismic response for the hard rock site. The coarse (NI20) model, as described in Subsection 3G.2.2.2, is used for the soil structure interaction (SSI) analyses and is set up in both ANSYS and SASSI. The NI05 model, as described in Appendix 3G.2.2.4, is used to develop amplified seismic response for the envelope of soil profiles presented in Subsection 3.7.1.4 for flexible regions not captured by the coarser NI20 model. The models and analyses are described in Appendix 3G.

For the hard rock site, the soil-structure interaction effect is negligible. Therefore, for the hard rock site, the nuclear island is analyzed as a fixed-base structure, using computer program ANSYS without the foundation media. The three components of earthquake (two horizontal and one vertical time histories) are applied simultaneously in the analysis. Since the NI10 finite element model of the auxiliary and shield building uses shell elements to represent the 6-foot-thick basemat, the nodes of the basemat element are at the center of the basemat (elevation 63′-6″). The finite element model of the containment internal structures uses solid elements, which extend down to elevation 60′-6″. When the finite element models are combined and used in the time history analyses, the auxiliary building finite element model is fixed at the shell element basemat nodes (elevation 63′-6″) and the base of the containment internal structures is fixed at the bottom of the solid element base nodes (elevation 60′-6″). This difference in elevation of the base fixity is not significant since the concrete between elevations 60′-6″ and 63′-6″, below the auxiliary building, is nearly rigid. There is no lateral

support due to soil or hard rock below grade. This case results in higher response than a case analyzed with full lateral support below grade.

For additional information on the method used to calculate displacement, see Appendix 3G.4.1 and Appendix 3G.4.2.

3.7.2.1.3 Response Spectrum Analysis

Response spectral analysis is used for the evaluation of the nuclear island structures. Response spectrum analyses are used to perform an analysis of a particular structure or portion of structure using the procedures described in Appendix 3G.4.3.1 and Subsections 3.7.2.6, 3.7.2.7, and 3.7.3.

Seismic response spectrum analysis of the auxiliary building, shield building, and containment internal structure is performed to develop the seismic design loads for these buildings, and the loads generated include the amplified load due to flexibility and the distribution of this load to the surrounding structures.

3.7.2.2 Natural Frequencies and Response Loads

Modal analyses are performed for the shell and lumped-mass stick models of the seismic Category I structures on the nuclear island, as described in Appendix 3G. Seismic response spectra at the six key locations (Subsection 3.7.2) are given in Appendix 3G.

3.7.2.3 Procedure Used for Modeling

Based on the general plant arrangement, three-dimensional, finite element models are developed for the nuclear island structures: a finite element model of the coupled shield and auxiliary buildings, a finite element model of the containment internal structures, a finite element model of the shield building roof, and an axisymmetric shell model of the steel containment vessel. These three-dimensional, finite element models provide the basis for the development of the dynamic model of the nuclear island structures.

The finite element models of the coupled shield and auxiliary buildings, and the containment internal structures are based on the gross concrete section with the modulus based on the specified compressive strength of concrete reduced by a factor of 0.8 to consider the effect of cracking as recommended in Table 6-5 of FEMA 356 (Reference 5). This 80-percent value is supported by non-linear ABAQUS analyses performed on the nuclear island finite element model. The comparison between linear and non-linear models shows that the 80-percent stiffness model response spectra enveloped the non-linear model, providing a conservative approach in terms of response spectra and maximum stresses obtained in the shield building wall.

Seismic subsystems coupled to the overall dynamic model of the nuclear island include the coupling of the reactor coolant loop model to the model of the containment internal structures, and the coupling of the polar crane model to the model of the steel containment vessel. The criteria used for decoupling seismic subsystems from the nuclear island model are according to Section II.3.b of SRP 3.7.2, Revision 2. The total mass of other major subsystems and equipment is less than one percent of the respective supporting nuclear island structures; therefore, the mass of other major subsystems and equipment is included as concentrated lumped-mass only.

Several minor (basic building configuration not modified) design changes and model improvements include the following:

- Provision for heavier fuel racks in the spent fuel pool area. Fuel and rack masses are updated, and pool water volumes are modeled as lumped masses.

- Changes in the annulus configuration are incorporated into the dish model, and lower shield building and upper containment internal structures basemat nodes and elements are modified for compatibility.

- The core makeup tanks were added as stick models.

- The polar crane model has a reduced weight and updated steel containment vessel local stiffness, and now includes polar crane truck stiffness.

The seismic analysis of the water inside the PCCWST was performed for the AP600. It was concluded that the low-frequency sloshing mode is not significant to the response of the nuclear island away from the shield building roof and that this conclusion could be extended to the AP1000 design. Further analysis indicated that the sloshing mass ratio remained essentially unchanged between the AP600 and AP1000.

3.7.2.3.1 Coupled Shield and Auxiliary Buildings and Containment Internal Structures

The finite element models of the coupled shield and auxiliary buildings and the reinforced concrete portions of the containment internal structures are based on the gross concrete section with the modulus based on the specified compressive strength of concrete of contributing structural walls and slabs. The properties of the concrete-filled structural modules are computed using the combined gross concrete section and the transformed steel face plates of the structural modules. The modulus is reduced by a factor of 0.8 to consider the effect of cracking. Furthermore, the weight density of concrete plus the uniformly distributed miscellaneous dead weights are considered by adding surface mass or by adjusting the material mass density of the structural elements. An equivalent tributary slab area load of 50 pounds per square foot is considered to represent miscellaneous deadweight such as minor equipment, piping and raceways. 25 percent of the floor live load or 75 percent of the roof snow load, whichever is applicable, is considered as mass in the global seismic models.

Major equipment weights are distributed over the floor area or are included as concentrated lumped masses at the equipment locations. The major equipment supported by the containment internal structures is represented by stick models connected to the containment internal structures, and includes the reactor coolant loop, the pressurizer, and the core makeup tank. The core makeup tank model is used only in the nuclear island fine (NI10) model; the core makeup tank is represented by mass in the nuclear island coarse model (NI20). The finite element models of the coupled shield and auxiliary buildings and the containment internal structures are described in Appendix 3G. The auxiliary and shield building is modeled with shell elements and the base of the finite element model is at the middle of the basemat at elevation 63′-6″. The bottom of the containment and internal structures are modeled with solid elements and the base of the finite element model is at the underside of the basemat at elevation 60′-6″. The interface between the models is at a radius of 71′-0″ at the mid-surface of the shield building.

3.7.2.3.2 Steel Containment Vessel

The steel containment vessel is a freestanding, cylindrical, steel shell structure with ellipsoidal upper and lower steel domes. The three-dimensional, lumped-mass stick model of the steel containment vessel is developed based on the axisymmetric shell model. Figure 3G.2-4 presents the steel containment vessel stick model. In the stick model, the properties are calculated as follows:

- Members representing the cylindrical portion are based on the properties of the actual circular cross section of the containment vessel.

- Members representing the bottom head are based on equivalent stiffnesses calculated from the shell of revolution analyses for static 1.0g in vertical and horizontal directions.

- Shear, bending and torsional properties for members representing the top head are based on the average of the properties at the successive nodes, using the actual circular cross section. These are the properties that affect the horizontal modes. Axial properties, which affect the vertical modes, are based on equivalent stiffnesses calculated from the shell of revolution analyses for static 1.0g in the vertical direction.

The equivalent static acceleration analyses of the containment vessel use a finite element shell model with a refined mesh in the area adjacent to the large penetrations. Comparison of this with a time history analysis for the regions immediately surrounding the large penetrations verifies that the loads from equivalent static analysis are conservative to time history using a representative study.

This method used to construct a stick model from the axisymmetric shell model of the containment vessel is verified by comparison of the natural frequencies determined from the stick model and the shell of revolution model as shown in Table 3G.2-2. The shell of revolution vertical model (n = 0 harmonic) has a series of local shell modes of the top head above elevation 265′ between 23 and 30 hertz. These modes are predominantly in a direction normal to the shell surface and cannot be represented by a stick model. These local modes have small contribution to the total response to a vertical earthquake as they are at a high frequency where seismic excitation is small. The only seismic Category I components attached to this portion of the top head are the water distribution weirs of the passive containment cooling system. These weirs are designed such that their fundamental frequencies are outside the 23 to 30 hertz range of the local shell modes.

Additional details of the steel containment vessel stick model are included in Appendix 3G.2.1.3.

The containment air baffle, presented in Subsection 3.8.4.1.3, is supported from the steel containment vessel at regular intervals so that a gap is maintained for airflow. It is constructed with individual panels which do not contribute to the stiffness of the containment vessel. The fundamental frequency of the baffle panels and supports is about twice the fundamental frequency of the containment vessel. The mass of the air baffle is small, equal to approximately 10 percent of the vessel plates to which it is attached. The air baffle, therefore, is assumed to have negligible interaction with the steel containment vessel. Only the mass of the air baffle is considered and added at the appropriate elevations of the steel containment vessel stick model.

The interaction between the polar crane and the containment vessel is significant and is included in the model. This polar crane model reflects the polar crane wheel assemblies. The polar crane is supported on a ring girder which is an integral part of the steel containment vessel at elevation 228′-0″ as shown in Figure 3.8.2-1. It is modeled as a multi-degree of freedom system attached to the steel containment shell at elevation 224′ (midpoint of ring girder) as shown in Figure 3G.2-4. The polar crane is modeled as shown in Figure 3G.2-5 with five masses at the mid-height of the bridge at elevation 233′-6″ and one mass for the trolley. The polar crane model includes the flexibility of the crane bridge girders and truck assembly, and the containment shell's local flexibility. When fixed at the center of containment, the model shows fundamental frequencies of 3.7 hertz transverse to the bridge, 6.4 hertz vertically, and 8.5 hertz along the bridge.

[*During plant operating conditions, the polar crane is parked in the plant north-south direction with the trolley located at one end near the containment shell.*]* In the seismic model, the crane bridge spans in the north-south direction and the mass eccentricity of the trolley is considered by locating the mass of the trolley at the northern limit of travel of the main hook. Furthermore, the mass eccentricity of the two equipment hatches and the two personnel airlocks are considered by placing their mass at their respective center of mass as shown in Figure 3G.2-4. Any modeling change due to the as-procured crane data is resolved with the COL holder item in Subsection 3.7.5.4, “Reconciliation of Seismic Analyses of Nuclear Island Structures.”

*NRC Staff approval is required prior to implementing a change in this information.

3.7.2.3.3 Nuclear Island Seismic Model

The nuclear island seismic models are described in Appendix 3G. The various building models are interconnected to form the overall dynamic model of the nuclear island. The mass properties of the models include all tributary mass expected to be present during plant operating conditions. This includes the dead weight of walls and slabs, weight of major equipment, and equivalent tributary slab area loads representing miscellaneous equipment, piping and raceways.

The hydrodynamic mass effect of the water within the passive containment cooling system water tank on the shield building roof, the in-containment refueling water storage tank within the containment internal structures, and the spent fuel pool in the auxiliary building is evaluated. Since the water in the PCCS tank responds at a very low frequency (sloshing) and does not affect building response, the PCCS tank water horizontal mass is reduced to exclude the low frequency water sloshing mass. The total mass of the water in the in-containment refueling water storage tank within the containment internal structures, and the spent fuel pool in the auxiliary building is included in the nuclear island seismic model.

Seismic response spectra are developed at the locations of the nodes. These response spectra are grouped and enveloped to define the seismic design response spectra. The nodes associated with a specific elevation and building structure (i.e., auxiliary and shield building and containment internal structures) are grouped. For the auxiliary and shield building where the floor at the elevation of interest is rigid (i.e. frequency > 33 hertz), it is only necessary to envelop the response spectra at edge points and interior nodes at the shield wall to obtain the largest seismic response spectra because of rigid motion. The edge nodes reflect the largest rocking and translational response of the auxiliary building, and the response spectra associated with the nodes on the shield wall will reflect the shield wall dynamic response. It is not necessary to include any nodes between the shield wall and auxiliary building edge since the floor is rigid, and the response cannot be worse than those enveloped.

A refined finite element shell model of the nuclear island concrete structures is reviewed for flexible regions, which may produce amplified response spectra. This model, called the NI05 model, has a tetrahedral mesh size of approximately 5 feet by 5 feet. Each of the principal walls and floors in the auxiliary and shield building as well as the containment internal structures are reviewed. A modal analysis of the NI05 model for both auxiliary and shield building and containment internal structures is reviewed for each of these regions for the existence of out of plane modes, which are considered flexible (less than 33 hertz) with significant participating mass. The survey reveals that some regions, typically in the middle of a floor or wall, exhibit amplified behavior compared to the critical nodes at the corner and edge building locations. [*These regions, which have flexible areas, are evaluated in one of two ways:*

- *Flexible areas, which have been previously identified, have amplified response spectra developed directly from the time history analyses for the envelope of soil sites.*

- *Flexible regions, which require a detailed analysis to obtain the amplified response spectra, use input directly from time history analysis. The NI05 finite element model is used to capture out-of-plane flexibilities that, because of mesh refinement, a more course model could not capture.*

If equipment or a structure is supported at more than one elevation, then the seismic input as an envelope of multiple groups based on the support locations will be defined. Therefore, if the equipment or structure is supported on rigid and flexible floor areas the response spectra (horizontal and vertical directions) used by the analysts will be the envelope of the rigid and flexible areas that include inside and outside nodes.

*NRC Staff approval is required prior to implementing a change in this information.

If an equipment or structure is supported exclusively by a floor or wall, only that spectra will be used for design.]*

3.7.2.4 Soil-Structure Interaction

Soil-structure interaction is not significant for the nuclear island founded on rock with a shear wave velocity greater than 8000 feet per second. The soil-structure interaction analyses for the firm rock and soil sites are described in Appendix 3G.

The computer program SASSI is used to perform the soil-structure interaction analysis. The SASSI model of the nuclear island is based on the NI20 Coarse Finite Element model. Soil-structure interaction analyses are performed based on the nuclear island 3D SASSI model for the three soil conditions established from the AP1000 2D SASSI analyses, in addition to soft rock and soft soil.

SASSI uses key frequencies to perform its transfer function calculations. For a large model, resting on a very stiff soil (hard rock), SASSI gives conservative results at high frequencies. The significant responses for AP1000 soil cases occur at less than 10 hertz so the SASSI model is adequate for use.

Analyses are performed with large solid-shell finite element models at two levels. The fine (NI10) model is used to define the seismic response for the hard rock site. The coarse (NI20) model is used for the soil-structure interaction analyses. The NI20 coarse model has fewer nodes and elements than the NI10 model. It captures the essential features of the nuclear island configuration. The nominal shell and solid element dimension is about 20 feet.

3.7.2.5 Development of Floor Response Spectra

The design floor response spectra are generated according to Regulatory Guide 1.122.

Seismic floor response spectra are computed using time-history responses determined from the nuclear island seismic analyses. The time-history responses for the hard rock condition are determined from a mode superposition time history analysis using computer program ANSYS.

The time-history responses for the firm rock and soil conditions are determined from a complex frequency response analysis using computer program SASSI. Floor response spectra for damping values equal to 2, 3, 4, 5, 7, 10, and 20 percent of critical damping are computed at the required locations.

The floor response spectra for the design of subsystems and components are generated by broadening the enveloped nodal response spectra determined for the hard rock site and soil sites.

The spectral peaks are broadened by ±15 percent to account for the variation in the structural frequencies, due to the uncertainties in parameters such as material and mass properties of the structure and soil, damping values, seismic analysis technique, and the seismic modeling technique. Figure 3.7.2-14 shows the broadening procedure used to generate the design floor response spectra. Spectral peaks at frequencies associated with fundamental soil structure interaction frequencies are reviewed. If there is a “valley” between peaks due to different soil profiles and not the building modal response, then this valley is filled by extending the broadening of the lower peak horizontally until it meets the broadened upper peak.

Floor response spectra for the auxiliary building are obtained from the three-dimensional model as described in Appendix 3G. These spectra are developed for the specific location in the auxiliary building. Where spectra at a number of nodes have similar characteristics, a single set of spectra may be developed by enveloping the broadened spectra at each of the nodes.

*NRC Staff approval is required prior to implementing a change in this information.

The safe shutdown earthquake floor response spectra for 5 percent damping, at representative locations of the coupled auxiliary and shield buildings, the steel containment vessel, and the containment internal structures are presented in Appendix 3G.

3.7.2.6 Three Components of Earthquake Motion

Seismic system analyses are performed considering the simultaneous occurrences of the two horizontal and the vertical components of earthquake.

In mode superposition time-history analyses using computer program ANSYS, the three components of earthquake are applied either simultaneously or separately. In the ANSYS analyses with the three earthquake components applied simultaneously, the effect of the three components of earthquake motion is included within the analytical procedure so that further combination is not necessary.

In analyses with the earthquake components applied separately and in the response spectrum and equivalent static analyses, the effect of the three components of earthquake motion are combined using one of the following methods:

- For seismic analyses with the statistically independent earthquake components applied separately, the time-history responses from the three earthquake components are combined algebraically at each time step to obtain the combined response time-history. This method is used in the SASSI analyses.

- The peak responses due to the three earthquake components from the response spectrum and equivalent static analyses are combined using the square root of the sum of squares (SRSS) method.

- The peak responses due to the three earthquake components from the equivalent static analyses are combined directly, using the assumption that when the peak response from one component occurs, the responses from the other two components are 40 percent of the peak (100 percent-40 percent-40 percent method). Combinations of seismic responses from the three earthquake components, together with variations in sign (plus or minus), are considered. This method is used in the nuclear island basemat analyses, the containment vessel analyses, and the shield building roof analyses.

The containment vessel is analyzed using axisymmetric finite element models. These axisymmetric building structures are analyzed for one horizontal seismic input from any horizontal direction and one vertical earthquake component. Responses are combined by either the square root of the sum of squares method or by the 100 percent-40 percent-40 percent method in which one component is taken at 100 percent of its maximum value and the other components are taken at 40 percent of their maximum value.

For the seismic responses presented in Appendix 3G, the effect of three components of earthquake are considered as follows:

- Mode Superposition Time History Analysis (program ANSYS) and the Complex Frequency Response Analysis (program SASSI) – the time history responses from the three components of earthquake motion are combined algebraically at each time step.

A summary of the dynamic analyses performed and the combination techniques used are presented in Appendix 3G.

3.7.2.7 Combination of Modal Responses

The modal responses of the response spectrum system structural analysis are combined using the procedures described in Appendix 3G.4.3. In the fixed base mode superposition time history analysis of the hard rock site, the total seismic response is obtained by superposing the modal responses within the analytical procedure so that further combination is not necessary.

A summary of the dynamic analyses performed and the combination techniques used are presented in Appendix 3G.

3.7.2.8 Interaction of Seismic Category II and Nonseismic Structures with Seismic Category I Structures, Systems, or Components

Nonseismic structures are evaluated to determine that their seismic response does not preclude the safety functions of seismic Category I structures, systems or components. This is accomplished by satisfying one of the following:

- The collapse of the nonseismic structure will not cause the nonseismic structure to strike a seismic Category I structure, system or component.
- The collapse of the nonseismic structure will not impair the integrity of seismic Category I structures, systems or components.
- The structure is classified as seismic Category II and is analyzed and designed to prevent its collapse under the safe shutdown earthquake.

The structures adjacent to the nuclear island are the annex building, the radwaste building, and the turbine building.

Category II/I interaction between the nuclear island and the annex building, radwaste building, and turbine building is documented in Subsections 3.7.2.8.1, 3.7.2.8.2, and 3.7.2.8.3.

3.7.2.8.1 Annex Building

The portion of the annex building adjacent to the nuclear island is classified as seismic Category II. The structural configuration is shown in Figure 3.7.2-19. The annex building is analyzed for the safe shutdown earthquake for the six soil profiles described in Subsection 3.7.1.4. For the hard rock site, a range of soil properties is assumed for the layer above rock at the level of the nuclear island foundation. Seismic input is defined by response spectra applied at the base of a dynamic model of the annex building. The seismic response spectra input at the base of the annex building are the envelopes of the range of soil sites and also envelope the AP1000 design free field ground spectra shown in Figures 3.7.1-1 and 3.7.1-2. The envelope of the maximum building response acceleration values is applied as equivalent static loads to a more detailed static model. See Subsection 3.7.2.8.4 for more discussion of modeling and seismic analysis.

The minimum space required between the annex building and the nuclear island to avoid contact is obtained by absolute summation of the deflections of each structure obtained from either a time history or a response spectrum analysis for each structure. The maximum displacement of the roof of the annex building is 1.6 inches in the east-west direction. The minimum clearance between the structural elements of the annex building above grade and the nuclear island is 4 inches.

The potential impact of Seismic Category II and non-seismic structures on Seismic Category I structures considering a postulated void condition under the Category II or non-seismic structure is

discussed in Subsection 3.7.2.8.3. The turbine building is considered to be the most critical building given the magnitude of the static and seismic loads.

3.7.2.8.2 Radwaste Building

The radwaste building is classified as nonseismic and is designed to the seismic requirements of the Uniform Building Code, Zone 2A with an Importance Factor of 1.25. As shown in the radwaste building general arrangement in Figure 1.2-22, it is a small steel framed building. If it were to impact the nuclear island or collapse in the safe shutdown earthquake, it would not impair the integrity of the reinforced concrete nuclear island. The minimum clearance between the structural elements of the radwaste building above grade and the nuclear island is 4 inches.

Three methods are used to demonstrate that a potential radwaste building impact on the nuclear island during a seismic event will not impair its structural integrity:

- The maximum kinetic energy of the impact during a seismic event considers the maximum radwaste building and nuclear island velocities. The total kinetic energy is considered to be absorbed by the nuclear island and converted to strain energy. The deflection of the nuclear island is less than 0.2″. The shear forces in the nuclear island walls are less than the ultimate shear strength based on a minus one standard deviation of test data.

- Stress wave evaluation shows that the stress wave resulting from the impact of the radwaste building on the nuclear island has a maximum compressive stress less than the concrete compressive strength.

- An energy comparison shows that the kinetic energy of the radwaste building is less than the kinetic energy of tornado missiles for which the exterior walls of the nuclear island are designed.

The potential impact of Seismic Category II and non-seismic structures on Seismic Category I structures considering a postulated void condition under the Category II or non-seismic structure is discussed in Subsection 3.7.2.8.3. The turbine building is considered to be the most critical building given the magnitude of the static and seismic loads.

3.7.2.8.3 Turbine Building

The south end of the turbine building is separated from the rest of the turbine building by a 2'-0" thick reinforced concrete wall that provides a robust structure around the first bay. This wall isolates the first bay of the turbine building from the general area of the turbine building and from the adjacent yard area. The main segment of this wall is located on column line 11.2. This wall extends from El.100'-0" basemat to the El.161'-0" operating floor. The first bay of the turbine building is classified as seismic Category II. The other bays are classified as non-seismic. The structure configuration is shown in Figure 3.7.2-20.

The first bay of the turbine building is analyzed for the safe shutdown earthquake for the six soil profiles described in Subsection 3.7.1.4. For the hard rock site, a range of soil properties is assumed for the layer above rock at the level of the nuclear island foundation. Seismic input is defined by response spectra applied at the base of a dynamic model of the first bay of the turbine building. The seismic response spectra input at the base of the first bay of the turbine building are the envelopes of the range of soil sites and also envelope the AP1000 design free field ground spectra shown in Figures 3.7.1-1 and 3.7.1-2. See Subsection 3.7.2.8.4 for more discussion of modeling and seismic analysis.

The first bay is designed in accordance with ACI-349 for concrete features and AISC-N690 for steel features.

For the non-seismic portion of the Turbine Building, seismic design is upgraded from Zone 2A, Importance Factor of 1.25, to Zone 3 with an Importance Factor of 1.0 in order to provide margin against collapse during the safe shutdown earthquake. The turbine building is an eccentrically braced steel frame structure designed to meet the following criteria:

- The turbine building is designed in accordance with ACI-318 for concrete structures and with AISC for steel structures. Seismic loads are defined in accordance with the 1997 Uniform Building Code provisions for Zone 3 with an Importance Factor of 1.0. For an eccentrically braced structure the resistance modification factor is 7 (UBC-97, reference 1) using strength design. When using allowable stress design, the allowable stresses are not increased by one third for seismic loads and the resistance modification factor is increased to 10 (UBC-91).

- The design of the lateral bracing system complies with the seismic requirements for eccentrically braced frames given in section 9.3 of the AISC Seismic Provisions for Structural Steel Buildings (Reference 34). Quality assurance is in accordance with ASCE 7-98 (Reference 35) for the lateral bracing system.

Although large voids and karst features are not considered likely at the site, as described in Appendix 2.5AA, a sensitivity analysis has been conducted to evaluate the safety of these buildings under postulated void conditions that are similar to those considered for the nuclear island. According to this, a tunnel-shaped (i.e., cylindrical) void with a 20-foot diameter is considered underneath the Category II and non-seismic structures with the top of the void at El. -60 feet. For this postulated void condition, the most critical building is considered to be the turbine building given the magnitude of the static and seismic loads for this building.

The sensitivity analyses reported here consider extremely unlikely and conservative cases that are only assessed to show the safety margin provided by the rock mass; these cases are not likely and are not for design purposes.

For the analysis reported here, the 3D finite element model (as presented in Subsections 2.5.4.10.3.2 and 2.5.4.10.8) is updated for the void case presented above. Best estimate material properties (FD1 properties for rock layers) are used for this analysis, as described in Subsection 2.5.4.10.3.

The postulated void is assumed to be water-filled, and is therefore modeled with the same pore pressure distribution (phreatic surface) as the surrounding rock.

The model considers a construction sequence that includes the following activities:

- Initial gravity loading (without the void),
- Gravity loading (with the void),
- Dewatering,
- Excavation and fill placement,
- Loading,
- Rewatering,

- Pseudo-Dynamic Loading (Multiplier of 1), and
- Pseudo-Dynamic Loading (Multiplier of 2).

The void is not considered in the initial gravity loading phase because it would have developed over time; further, inserting the void in the second phase allows for an evaluation of any points reaching Mohr-Coulomb failure due to the presence of the void independent from the other construction activities.

Figure 3.7-203 shows the PLAXIS 3D model. The 3D mesh is refined to the extent possible in the area surrounding the void. The total number of elements is 104,760.

Pseudo-Dynamic

To consider the impact of the potential voids under dynamic conditions, dynamic bearing pressures from the SASSI model are converted to equivalent (approximately) static loads and applied to the PLAXIS 3D model.

The forces from the dynamic bearing are pressures distributed uniformly over areas of the northern (maximum uplift) and southern (maximum compression) halves of the turbine building. The maximum uplift bearing pressure as obtained from the upper bound, lower bound, and best estimate cases are applied on the northern half of the turbine building, whereas the maximum compressive bearing pressure as obtained from the upper bound, lower bound, and best estimate cases are applied on the southern half of the turbine building, such that the maximum overturning moment is applied in the north-south direction of the turbine building (therefore overturning towards the nuclear island).

This approach is very conservative because maximum compressive pressures and tensile pressures are applied at the same time to maximize the overturning moment.

Additionally, a case is considered where the load combinations are multiplied by a safety factor of 2. Table 3.7-202 shows the total loads considered (static and pseudo-dynamic). The sum of the static load and the seismic uplift pressure is negative if the overall pressure is compressive.

Response under Dynamic Loads

All model results presented are for the case with a tunnel-shaped (i.e., cylindrical) void with a 20-foot diameter circular cross-section. The tunnel-shaped (i.e., cylindrical) void is considered to be more critical than a smaller 20-foot diameter spherical void, or a distribution of spherical voids.

Yield at any point is considered to occur if the stress conditions reach the Mohr-Coulomb failure envelope. Under pseudo-dynamic loading (multiplier of 1 and multiplier of 2) there are no plastic points or tension points near the void location, indicating that the rock mass surrounding the void is not experiencing compressive failure according to Mohr-Coulomb failure envelope or tensile failure.

Another useful parameter to consider is the relative shear stress, which is a measure to define how close the stress condition is to the Mohr-Coulomb failure envelope. Relative shear stresses are defined in Equation 3.7.2-1 (Reference 203).

$$\tau_{rel} = \frac{\tau_{mob}}{\tau_{max}} \qquad \text{Equation 3.7.2-1}$$

Where,

τ_{rel} = relative shear stress,

τ_{mob} = mobilized shear strength (maximum value of shear stress), and

τ_{max} = maximum value of shear stress for the case where the Mohr's circle is expanded to touch the Coulomb failure envelope while keeping the center of Mohr's circle constant.

Based on Equation 3.7.2-1, relative shear stresses provide a measure of margin compared to Mohr-Coulomb failure. For example, if the relative shear stress is equal to 1, then that location is marked with a plastic point. If the relative shear stress is much less than 1, the point is not close to the Mohr-Coulomb failure envelope. As shown by Figures 3.7-204 and 3.7-205, under pseudo-dynamic loading (multiplier of 1 and multiplier of 2) the rock surrounding the void indicates relative shear stresses much less than 1.

In conclusion, the presence of a 20-foot wide tunnel as modeled here does not present stability concerns under pseudo-dynamic loading (multiplier of 1 and multiplier of 2), i.e., no subsurface collapse is anticipated.

As indicated by the results described above and shown in Figures 3.7-204 and 3.7-205:

- No plastic points or tension points are observed during the pseudo-dynamic loading conditions, and

- The rock surrounding the void indicates relative shear stresses much less than 1.

The 20-foot diameter tunnel (cylindrical) void has been demonstrated to not be critical to the pseudo-dynamic stability of the turbine building. The turbine building is considered to be the most critical building given the magnitude of the static and seismic loads. The tunnel-shaped (i.e., cylindrical) void with a 20-foot diameter circular cross-section is considered to be more critical than a smaller 20-foot diameter spherical void, or a distribution of spherical voids.

In summary, the void size considered has been demonstrated to not be critical to the pseudo-dynamic stability of Category II or non-seismic structures. In other words, subsurface collapse is not anticipated under the combination of static and seismic Category II or non-seismic building loads. Therefore, it has been demonstrated that the Category II or non-seismic structures will not collapse due to the presence of the void considered and there will not be interaction between the Category II or non-seismic structures and the Category I structure.

3.7.2.8.4 Seismic Modeling and Analysis of Seismic Category II Building Structures

Seismic Category II structures, systems, and components are designed so that the safe shutdown earthquake does not cause unacceptable structural failure or interaction with seismic Category I items. Therefore, the seismic response of seismic Category II buildings must be obtained so that they can be designed to meet the seismic Category II requirements as given in Subsection 3.2.1.1.2. Seismic Category II structures are analyzed and evaluated in the same manner as seismic Category I structures. The foundation of the non-seismic portion is modeled with the associated mass distributed on it so that the soil structure interaction during a seismic event is reflected in the analysis.

The seismic analyses performed for the adjacent seismic Category II structures are simulated 3D analyses. The seismic analyses are performed primarily using 2D SASSI models. To properly account for the 3D effect, the response from 2D and 3D SASSI analyses of the seismic Category II buildings on rigid foundations are compared and a 3D effects factor is developed from this comparison. Three soil cases (upper bound soft to medium UBSM, soft to medium SM, and soft soil SS) are used to determine the 3D factor. Shown in Figures 3.7.2-20 and 3.7.2-21 are the 2D SASSI models with adjacent building structures. The seismic Category II buildings are modeled as stick models. The 3D model with adjacent structures is shown in Figure 3.7.2-22.

Seismic Category II buildings are designed using envelope foundation input response spectra (FIRS). The development of these FIRS shall be based on a number of analyses results from the SASSI analyses. The seismic Category II FIRS shall be the envelope of the SASSI seismic Category II foundation response spectra resulting from the following seismic inputs/soil profiles:

- AP1000 CSDRS – Hard rock at El. 60.5′.
- AP1000 CSDRS – Firm rock, soft rock, upper bound soft to medium, soft to medium, and soft soil soil profiles with AP1000 CSDRS spectra input at plant grade; and
- AP1000 hard rock high frequency (HRHF) – For rock sites, HRHF at plant grade shall be developed using AP1000 HRHF spectra at El. 60.5′ and a range of backfill soil profiles. The backfill soil under the annex and turbine buildings has a parabolic soil profile as a function of depth (El. 100′ to El. 60.5′) and uses EPRI (1993) strain dependent curves. The HRHF at plant grade spectrum shall be generated using soil profiles corresponding to a shear wave velocity of 500 fps, 750 fps, and 1000 fps at El. 100′. The HRHF at plant grade shall be used as input to SASSI analyses to determine the FIRS at the base of the seismic Category II structures.

For each soil case, 2D SASSI analyses shall be performed and the results at three locations at the base of the seismic Category II structures are enveloped. The maximum bearing demand and maximum relative displacement shall be established from the 2D SASSI analyses. The 3D effect factor is applied to the envelope foundation spectra and used for the design of the annex building and turbine building first bay.

Response spectrum analyses (using detailed finite element building models) shall be used to obtain seismic design loads for the seismic Category II building design. The seismic input to the response spectrum analyses is the envelope foundation response spectra obtained from the SASSI analyses. The COL applicant will perform the following screening criteria to determine if it has to perform further analysis for its site. If the requirements given below are not met, then the site applicant can perform site-specific analyses to demonstrate that its site-specific seismic Category II foundation seismic response spectra are less than the AP1000 annex building and turbine building first bay generic design envelope foundation spectra.

- The site meets Subsection 2.5.4.2 soil uniformity requirements.
- For soil sites, the site GMRS is enveloped by the AP1000 CSDRS with soil profiles SS, SM, UBSM, SR, FR, and HR.
- For HRHF sites, the site GMRS is enveloped by the AP1000 HRHF response spectra with a minimum backfill surface shear wave velocity of 500 fps, and a minimum lateral extent of the backfill corresponding to a line extending down from the surface at a one horizontal to one vertical (1H:1V) slope from the outside footprint limit of the seismic Category II structure.
- The bearing capacity with appropriate factor of safety is greater than or equal to the bearing demand.

3.7.2.9 Effects of Parameter Variations on Floor Response Spectra

Seismic model uncertainties due to, among other things, uncertainties in material properties, mass properties, damping values, the effect of concrete cracking, and the modeling techniques are accounted for in the widening of floor response spectra, as described in Subsection 3.7.2.5. The effect of cracking of the concrete-filled structural modules inside containment due to thermal loads is discussed in Subsection 3.8.3.4.2.

3.7.2.10 Use of Constant Vertical Static Factors

The vertical component of the safe shutdown earthquake is considered to occur simultaneously with the two horizontal components in the seismic analyses. Therefore, constant vertical static factors are not used for the design of seismic Category I structures.

3.7.2.11 Method Used to Account for Torsional Effects

The seismic analysis models of the nuclear island incorporate the mass and stiffness eccentricities of the seismic Category I structures and the torsional degrees of freedom.

For the response spectrum analysis of the nuclear island, the seismic loads are combined by means of the square root of the sum of the squares (SRSS). The equation for SRSS is shown below.

$$\sqrt{(\alpha \cdot A_{NS})^2 + (\alpha \cdot A_{EW})^2 + (A_{VT})^2}$$

where,

A_{NS} maximum element forces due to SSE response analysis in X (NS)

A_{EW} maximum element forces due to SSE response analysis in Y (EW)

A_{VT} maximum element forces due to SSE response analysis in Z (VT)

α factor to account for accidental torsion effect in NS or EW (1.05)

Alternatively, for equivalent static analysis, the 100-40-40 rule is applicable in order to cover both negative and positive member forces. The equation for the 100-40-40 rule is shown below.

$$k_1 \cdot \mathrm{sign}(A_{NS}) \cdot (\alpha \cdot A_{NS}) + k_2 \cdot \mathrm{sign}(A_{EW}) \cdot (\alpha \cdot A_{EW}) + k_3 \cdot A_{VT}$$

where,

k_i combination factors (±1.0, ±0.4, ±0.4)

sign(X) sign of variable X: $X < 0$ results -1; $X \geq 0$ results +1

α factor to account for accidental torsion effect in NS or EW (1.05)

3.7.2.12 Methods for Seismic Analysis of Dams

Seismic analysis of dams is site specific design.

There are no existing or new dams whose failure could affect the site interface flood level specified in Subsection 2.4.1.2, as discussed in Subsection 2.4.4.

3.7.2.13 Determination of Seismic Category I Structure Overturning Moments

Subsection 3.8.5.5.4 describes the effects of seismic overturning moments.

3.7.2.14 Analysis Procedure for Damping

Subsection 3.7.1.3 presents the damping values used in the seismic analyses. [*For structures comprised of different material types, the composite modal damping approach utilizing the strain energy method is used to determine the composite modal damping values.*]* Subsection 3.7.2.4 presents the damping values used in the soil-structure interaction analysis.

3.7.3 Seismic Subsystem Analysis

This subsection describes the seismic analysis methodology for subsystems, which are those structures and components that do not have an interface with the soil-structure interaction analyses. Structures and components considered as subsystems include the following:

- Structures, such as floor slabs, walls, miscellaneous steel platforms and framing
- Equipment modules consisting of components, piping, supports, and structural frames
- Equipment including vessels, tanks, heat exchangers, valves, and instrumentation
- Distributive systems including piping and supports, electrical cable trays and supports, HVAC ductwork and supports, instrumentation tubing and supports, and conduits and supports

Subsection 3.9.2 describes dynamic analysis methods for the reactor internals. Subsection 3.9.3 describes dynamic analysis methods for the primary coolant loop support system. Subsection 3.7.2 describes the analysis methods for seismic systems, which are those structures and components that are considered with the foundation and supporting media. Section 3.2 includes the seismic classification of building structures, systems, and components.

3.7.3.1 Seismic Analysis Methods

The methods used for seismic analysis of subsystems include, modal response spectrum analysis, time-history analysis, and equivalent static analysis. The methods described in this subsection are acceptable for any subsystem. The particular method used is selected by the designer based on its appropriateness for the specific item. Items analyzed by each method are identified in the descriptions of each method in the following paragraphs.

3.7.3.2 Determination of Number of Earthquake Cycles

Seismic Category I structures, systems, and components are evaluated for one occurrence of the safe shutdown earthquake (SSE). In addition, subsystems sensitive to fatigue are evaluated for cyclic motion due to earthquakes smaller than the safe shutdown earthquake. Using analysis methods, these effects are considered by inclusion of seismic events with an amplitude not less than one-third of the safe shutdown earthquake amplitude. The number of cycles is calculated based on IEEE-344-1987 (Reference 16) to provide the equivalent fatigue damage of two full safe shutdown earthquake events with 10 high-stress cycles per event. Typically, there are five seismic events with an amplitude equal to one-third of the safe shutdown earthquake response. Each of the one-third safe shutdown earthquake events has 63 high-stress cycles. [*For ASME Class 1 piping, the fatigue evaluation is performed based on five seismic events with an amplitude equal to one-third of the safe shutdown earthquake response. Each event has 63 high-stress cycles.*]*

When seismic qualification is based on dynamic testing for structures, systems, or components containing mechanisms that must change position in order to function, operability testing is performed for the safe shutdown earthquake preceded by one or more earthquakes. The number of preceding earthquakes is calculated based on IEEE-344-1987 (Reference 16) to provide the

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equivalent fatigue damage of one safe shutdown earthquake event. Typically, the preceding earthquake is one safe shutdown earthquake event or five one-half safe shutdown earthquake events.

3.7.3.3 Procedure Used for Modeling

The dynamic analysis of any complex system requires the discretization of its mass and elastic properties. This is accomplished by concentrating the mass of the system at distinct characteristic points or nodes, and interconnecting them by a network of elastic springs representing the stiffness properties of the systems. The stiffness properties are computed either by hand calculations for simple systems or by finite element methods for more complex systems.

Nodes are located at mass concentrations and at additional points within the system. They are selected in such a way as to provide an adequate representation of the mass distribution and high-stress concentration points of the system.

At each node, degrees of freedom corresponding to translations along three orthogonal axes, and rotations about these axes are assigned. The number of degrees of freedom is reduced by the number of constraints, where applicable. For equipment qualification, reduced degrees of freedom are acceptable provided that the analysis adequately and conservatively predicts the response of the equipment.

The size of the model is reviewed so that a sufficient number of masses or degrees of freedom are used to compute the response of the system. A model is considered adequate provided that additional degrees of freedom do not result in more than a 10 percent increase in response, or the number of degrees of freedom equals or exceeds twice the number of modes with frequencies less than 33 hertz.

Dynamic models of floor and roof slabs and miscellaneous steel platforms and framing include masses equal to 25 percent of the floor live load or 75 percent on the roof snow load, whichever is applicable.

Dynamic models are prepared for the following seismic Category I steel structures. Response spectrum or time history analyses are performed for structural design.

- Passive containment cooling valve room (room number 12701)
- Steel framing around steam generators
- Containment air baffle

Seismic input for the subsystem and component design are the enveloped floor response spectra described in Subsection 3.7.2.5 or the response time histories as described in Subsection 3.7.2.1. Where amplified response spectra are required on the subsystem for design of components, such as for use in the decoupled analyses of piping or components described in Subsection 3.7.3.8.3, the amplified response spectra are generated and enveloped as described in Subsection 3.7.2.5.

3.7.3.4 Basis for Selection of Frequencies

The effect of the building amplification on equipment and components is addressed by the floor response spectra method or by a coupled analysis of the building and equipment. Certain components are designed for a natural frequency greater than 33 hertz. In those cases where it is practical to avoid resonance, the fundamental frequencies of components and equipment are selected to be less than one-half or more than twice the dominant frequencies of the support structure.

3.7.3.5 Equivalent Static Load Method of Analysis

[*The equivalent static load method involves equivalent horizontal and vertical static forces applied at the center of gravity of various masses. The equivalent force at a mass location is computed as the product of the mass and the seismic acceleration value applicable to that mass location. Loads, stresses, or deflections, obtained using the equivalent static load method, are adjusted to account for the relative motion between points of support when significant.*]*

3.7.3.5.1 Single Mode Dominant or Rigid Structures or Components

For rigid structures and components, or for cases where the response can be classified as single mode dominant, the following procedures are used. Examples of these systems, structures, and components are equipment, and piping lines, instrumentation tubing, cable trays, HVAC, and floor beams modeled on a span by span basis.

- For rigid systems, structures, and components (fundamental frequency ≥ 33 hertz), an equivalent seismic load is defined for the direction of excitation as the product of the component mass and the zero period acceleration value obtained from the applicable floor response spectra.

- A rigid component (fundamental frequency ≥ 33 hertz), whose support can be represented by a flexible spring, can be modelled as a single degree of freedom model in the direction of excitation (horizontal or vertical directions). The equivalent static seismic load for the direction of excitation is defined as the product of the component mass and the seismic acceleration value at the natural frequency from the applicable floor response spectra. If the frequency is not determined, the peak acceleration from the applicable floor response spectrum is used.

[• *If the component has a distributed mass whose dynamic response will be single mode dominant, the equivalent static seismic load for the direction of excitation is defined as the product of the component mass and the seismic acceleration value at the component natural frequency from the applicable floor response spectra times a factor of 1.5. A factor of less than 1.5 may be used if justified. Static factors smaller than 1.5 are not used for piping systems.*]* A factor of 1.0 is used for structures or equipment that can be represented as uniformly loaded cantilever, simply supported, fixed-simply supported, or fixed-fixed beams (References 10 and 11) when the fundamental frequency is higher than the peak acceleration frequency associated with the applicable floor response spectrum. If the frequency is not determined, the peak acceleration from the applicable floor response spectrum is used.

3.7.3.5.2 Multiple Mode Dominant Response

This procedure applies to piping, instrumentation tubing, cable trays, and HVAC that are multiple span models. The equivalent static load method of analysis can be used for design of piping systems, instrumentation and supports that have significant responses at several vibrational frequencies. In this case, [*a static load factor of 1.5 is applied to the peak accelerations of the applicable floor response spectra. For runs with axial supports which are rigid in the axial direction (fundamental frequency greater than or equal to 33 hertz), the acceleration value of the mass of piping in its axial direction may be limited to 1.0 times its calculated spectral acceleration value. The spectral acceleration value is based on the frequency of the piping system along the axial direction. The relative motion between support points is also considered.*]*

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3.7.3.6 Three Components of Earthquake Motion

[*Two horizontal components and one vertical component of seismic response spectra are employed as input to a modal response spectrum analysis.*]* The spectra are associated with the safe shutdown earthquake. In the response spectrum and equivalent static analyses, the effects of the three components of earthquake motion are combined using one of the following methods:

[• *The peak responses due to the three earthquake components from the response spectrum analyses are combined using the square root of the sum of squares (SRSS) method.*

- *The peak responses due to the three earthquake components are combined directly, using the assumption that when the peak response from one component occurs, the responses from the other two components are 40 percent of the peak (100 percent-40 percent-40 percent method). Combinations of seismic responses from the three earthquake components, together with variations in sign (plus or minus), are considered. This method is not used for piping systems.*

One set of three mutually orthogonal artificial time histories is used when time-history analyses are performed. The components of earthquake motion specified in the three directions are statistically independent and applied simultaneously. When this method is used, the responses from each of the three components of motion are combined algebraically at each time step.]*

In addition, an optional method for combining the response of the three components of earthquake motion is presented in the following paragraphs.

[*The time-history safe shutdown earthquake analysis of a subsystem can be performed by simultaneously applying the displacements and rotations at the interface point(s) between the subsystem and the system. These displacements and rotations are the results obtained from a model of a larger subsystem or a system that includes a simplified representation of the subsystem. The time-history safe shutdown earthquake analysis of the system is performed by applying three mutually orthogonal and statistically independent, artificial time histories.*]* Possible examples of the use of this method of seismic analysis include the following:

- The subsystem analysis is a flexible floor or miscellaneous structural steel frame. The corresponding system analysis is the soil-structure interaction analysis of the nuclear island structures.
- The subsystem analysis is the primary loop piping system and interior concrete building structure. The interface point is the top of the basemat. The corresponding system analysis is the soil-structure interaction analysis of the nuclear island structures.
- The subsystem analysis is the reactor coolant pump and internal components. The interface points are the welds on the pump suction and discharge nozzles. The corresponding system analysis is the primary loop piping system and interior concrete building structure.

3.7.3.7 Combination of Modal Responses

[*For the seismic response spectra analyses, the zero period acceleration cut-off frequency is 33 hertz. High frequency or rigid modes are considered using the left-out-force method or the missing mass method*]* described in Subsection 3.7.3.7.1. The method to combine the low frequency modes is described in Subsection 3.7.3.7.2. [*The rigid mode results in the three perpendicular directions of the seismic input are combined by the SRSS method. The resultant response of the rigid modes is combined by SRSS with the flexible mode results.*]* The combination of modal responses in time

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history analyses of piping systems is described in Subsection 3.7.3.17 Modal responses in time history analyses of other subsystems are combined as described in Subsection 3.7.2.6.

3.7.3.7.1 Combination of High-Frequency Modes

This subsection describes alternative methods of accounting for high-frequency modes (generally greater than 33 hertz) in seismic response spectrum analysis. Higher-frequency modes can be excluded from the response calculation if the change in response is less than or equal to 10 percent.

3.7.3.7.1.1 Left-Out-Force Method or Missing Mass Correction for High Frequency Modes

The left-out-force method is based on the Left-Out-Force Theorem. This theorem states that for every time history load there is a frequency, f_r, called the "rigid mode cutoff frequency" above which the response in modes with natural frequencies above f_r will very closely resemble the applied load at each instant of time. These modes are called "rigid modes." [*The left-out-force method is used in program PIPESTRESS.*]*

The left-out-force vector, $\{Fr\}$, is calculated based on lower modes:

$$\{Fr\} = \left[1 - \Sigma M e_j e_j^T\right] f(t)$$

where:

f (t) = the applied load vector

M = the mass matrix

e_j = the eigenvector

Note that Σ is only for all the flexible modes, not including the rigid modes.

In the response spectra analysis, the total inertia force contribution of higher modes can be interpreted as:

$$\{Fr\} = Am[M]\left[\{r\} - \Sigma P_j e_j\right]$$

where:

Am = the maximum spectral acceleration beyond the flexible modes

[M] = the mass matrix

$\{r\}$ = the influence vector or displacement vector due to unit displacement

P_j = participation factor

Since,

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$$P_j = e_j^T [M]\{r\}, \{Fr\} = Am[M]\{r\}\left[1 - \Sigma M e_j e_j^T\right]$$

[*In PIPESTRESS, the low frequency modes are combined by one of the Regulatory Guide 1.92 methods in the response spectrum analysis.*]* For each support level, there is a pseudo-load vector or left-out-force vector in the X, Y and Z directions. These left-out-force vectors are used to generate left-out-force solutions which are multiplied by a scalar amplitude equal to a magnification factor specified by the user. This factor is usually the ZPA (zero period acceleration) of the response spectrum for the corresponding direction. The resultant low frequency responses are combined by square root of the sum of the squares with the high frequency responses (rigid modes results).

[*In GAPPIPE, the results from the high frequency responses are also combined by the square root of the sum of the squares with those from the resultant loads contributed by lower modes.*]* The missing mass correction for an independent support motion or multiple response spectra analysis is exactly the same as that for the single enveloped response spectrum analysis except that Am used is the envelope of all the zero period accelerations of all the independent support inputs.

3.7.3.7.1.2 SRP 3.7.2 Method

[*The method described in SRP Section 3.7.2 may also be used for combination of high-frequency modes.*]*

The following is the procedure for incorporating responses associated with high-frequency modes.

Step 1 Determine the modal responses only for those modes having natural frequencies less than that at which the spectral acceleration approximately returns to the zero period acceleration (33 hertz for the Regulatory Guide 1.60 response spectra). Combine such modes according to the methods discussed in Subsection 3.7.3.7.2.

Step 2 For each degree of freedom included in the dynamic analysis, determine the fraction of degree of freedom mass included in the summation of all modes included in Step 1. This fraction d_i for each degree of freedom is given by:

$$d_i = \sum_{n=1}^{N} C_n \text{ x } \phi_{n,i}$$

where:

n = order of mode under consideration

N = number of modes included in Step 1

$\phi_{n,i}$ = nth natural mode of the system

C_n is the participation factor given by:

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$$C_n = \frac{(\phi_n)^T [m](1)}{(\phi_n)^T [m](\phi_n)}$$

Next, determine the fraction of degree of freedom mass not included in the summation of these modes:

$$e_i = d_i - \delta_{ij}$$

where δ_{ij} is the Kronecker delta, which is 1 if degree of freedom i is in the direction of the earthquake motion and 0 if degree of freedom i is a rotation or not in the direction of the earthquake input motion.

If, for any degree of freedom i, the absolute value of this fraction e_i exceeds 0.1, the response from higher modes is included with those included in Step 1.

Step 3 Higher modes can be assumed to respond in phase with the zero period acceleration and, thus, with each other. Hence, these modes are combined algebraically, which is equivalent to pseudostatic response to the inertial forces from these higher modes excited at the zero period acceleration. The pseudostatic inertial forces associated with the summation of all higher modes for each degree of freedom i are given by:

$$P_i = ZPA \times M_i \times e_i$$

where:

P_i = force or moment to be applied by degree of freedom i

M_i = mass or mass moment of inertia associated with degree of freedom i.

The subsystem is then statically analyzed for this set of pseudo static inertial forces applied to all degrees of freedom to determine the maximum responses associated with high-frequency modes not included in Step 1.

Step 4 The total combined response to high-frequency modes (Step 3) is combined by the square root of sum of the squares method with the total combined response from lower-frequency modes (Step 1) to determine the overall structural peak responses.

3.7.3.7.2 Combination of Low-Frequency Modes

This subsection describes the method for combining modal responses in the seismic response spectra analysis. [*The total unidirectional seismic response for subsystems is obtained by combining the individual modal responses using the square root sum of the squares method. For subsystems having modes with closely spaced frequencies, this method is modified to include the possible effect of these modes. For piping systems, the methods in Regulatory Guide 1.92 are used for modal combinations.*]* For other subsystems, the methods in Regulatory Guide 1.92 or the following alternative methods may be used. [*The groups of closely spaced modes are chosen so that the differences between the frequencies of the first mode and the last mode in the group do not exceed 10 percent of the lower frequency.*

Combined total response for systems having such closely spaced modal frequencies is obtained by adding to the square root sum of squares of all modes the product of the responses of the modes

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in each group of closely spaced modes and coupling factor.]* This can be represented mathematically as:

$$R_T^{\,2} = \sum_{i=1}^{N} R_i^{\,2} + 2 \sum_{j=1}^{S} \sum_{k=Mj}^{Nj-1} \sum_{\ell=k+1}^{Nj} R_k \, R_\ell \, \varepsilon_{k\ell}$$

where:

R_T	=	total unidirectional response
R_i	=	absolute value of response of mode i
N	=	total number of modes considered
S	=	number of groups of closely spaced modes
	=	lowest modal number associated with group j of closely spaced modes
N_j	=	highest modal number associated with group j of closely spaced modes
$\varepsilon_{k\ell}$	=	coupling factor, defined as follows:

$$\varepsilon_{k\ell} = \left(1 + \frac{\left(w_k^{'} - w_\ell^{'}\right)^2}{\left(\beta_k^{'} \, w_k + \beta_\ell^{'} \, w_\ell\right)^2} \right)^{-1}$$

and,

$$w_k^{'} = w_k \, \left[1 - \left(\beta_k\right)^2\right]^{1/2}$$

$$\beta_k^{'} = \beta_k + \frac{2}{w_k \, t_d}$$

where:

w_k	=	frequency of closely spaced mode k
β_k	=	fraction of critical damping in closely spaced mode k
t_d	=	duration of the earthquake (= 30 seconds)

[*Alternatively, a more conservative grouping method can be used in the seismic response spectra analyses. The groups of closely spaced modes are chosen so that the difference between two frequencies is no greater than 10 percent.*]* Therefore,

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$$R_T{}^2 = \sum_{i=1}^{N} R_i{}^2 + 2\Sigma\, \varepsilon_{k\ell}\, R_k\, R_\ell$$

where:

$$\frac{|\, w_k - w_\ell \,|}{w_\ell} \leq 0.1$$

All other terms for the modal combination remain the same. The 10 percent grouping method is more conservative than the grouping method because the same mode can appear in more than one group.

In addition to the above methods, any of the other methods in Regulatory Guide 1.92 may be used for modal combination.

[*3.7.3.8 Analytical Procedure for Piping*

This subsection describes the modeling methods and analytical procedures for piping systems.

The piping system is modeled as beam elements with lump masses connected by a network of elastic springs representing the stiffness properties of the piping system. Concentrated weights such as valves or flanges are also modeled as lump masses. The effects of torsion (including eccentric masses), bending, shear, and axial deformations, and effects due to the changes in stiffness values of curved members are accounted for in the piping dynamic model.

The lump masses are selected so that the maximum spacing is not greater than the length that would produce a natural frequency equal to the zero period acceleration (ZPA) frequency of the seismic input when calculated based on a simply supported beam. As a minimum, the number of degrees of freedom is equal to twice the number of modes with frequencies less than the zero period acceleration frequency.

The piping system analysis model includes the effect of piping support mass when the contributory mass of the support is greater than 10 percent of the total mass of the effected piping spans. The contributory mass of the support is the portion of the support mass that is attached to the piping; such as clamps, bolts, trunnions, struts, and snubbers. Supports that are not directly attached to the piping, such as box frames, need not be considered for mass effects. The mass of the applicable support will not affect the response of the system in the supported direction, herefore only the unsupported direction needs to be considered. Based on this reasoning, the mass of full anchors can be neglected. The total mass of each effected piping span includes the mass of the piping, fluid contents, insulation, and any concentrated masses (for example, valves or flanges) between the adjacent supports in each unrestrained direction on both sides of the applicable support. For example; the contributory mass of an X direction support must be compared to the mass of the piping spans in the unrestrained Y and Z directions. A contributory support mass that is less than 10 percent of the masses of the effected spans will have insignificant effect on the response of the piping system and can be neglected.

The stiffness matrix of the piping system is calculated based on the stiffness values of the pipe elements and support elements. Default rigid or calculated support stiffness values are used (see subsections 3.9.3.1.5 and 3.9.3.4). When the support deflections are limited to 1/8 inches for the dynamic combined faulted loads, default rigid support stiffness values are used. If the dynamic combined faulted load deflection for any support exceeds 1/8 inches, calculated support stiffness values are used for the affected support.

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Valves, equipment and piping modules are considered as rigid if the natural frequencies are greater than 33 hertz. Valves with lower frequencies are included in the piping system model. See subsection 3.7.3.8.2.1 for flexible equipment and subsection 3.7.3.8.3 for flexible modules.

See subsection 3.9.3.1.4 for the primary loop piping and support system.]*

3.7.3.8.1 Supporting Systems

This subsection deals with the analysis of piping systems that provide support to other piping systems. The methods used for the analysis of the primary loop piping are described in Appendix 3C. [*The supported piping system may be excluded from the analysis of the supporting piping system when the ratio of the supported pipe to supporting pipe moment of inertia is less than or equal to 0.04.*

If the ratio of the run piping outside diameter to the branch piping outside diameter (nominal pipe size) exceeds or equals 3.0, the branch piping can be excluded from the analysis of the run piping. The mass and stiffness effects of the branch piping are considered as described below.

Stiffness Effect

The stiffness effect of the decoupled branch pipe is considered significant when the distance from the run pipe outside diameter to the first rigid or seismic support on the decoupled branch pipe is less than or equal to one half the deadweight span of the branch pipe (given in ASME III Code Subsection NF).

Mass Effect

Considering one direction at a time, the mass effect is significant when the weight of half the span (from the decoupling point) of the branch pipe in one direction is more than 20 percent the weight of the main run pipe span in the same direction. Concentrated weights in the branch pipe are considered. A branch pipe span in x direction is the span between the decoupled branch point and the first seismic or rigid support in the x direction. A main run pipe span in the x direction is the piping bounded by the first seismic or rigid support in the x direction on both sides of the decoupled branch point. Similarly, the same definition applies to the spans in other directions (y and z).

If the calculated branch pipe weight is less than 20 percent but more than 10 percent of the main run pipe weight, this weight is lumped at the decoupling point of the run pipe for the run pipe analysis. This weight can be neglected if it is less than 10 percent of the main run pipe weight.

Required Coupled Branch Piping

If the stiffness and/or mass effects are considered significant, the branch piping is included in the piping analysis for the run pipe analysis. The portion of branch piping considered in the analysis adequately represents the behavior of the run pipe and branch pipe. The branch line model ends in one of the following ways:

- *First six-way anchor*
- *Four rigid/seismic supports in each of the three perpendicular directions*
- *Rigidly supported zone as described* in *subsection 3.7.3.13.4.2*]*

3.7.3.8.2 Supported Systems

This subsection deals with the analysis of piping systems that are supported by other piping systems or by equipment.

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3.7.3.8.2.1 Large Diameter Auxiliary Piping

[*This subsection deals with ASME Class 1 piping larger than 1-inch nominal pipe size and ASME Class 2 and 3 piping with nominal pipe size larger than 2 inches. The response spectra methodology is used.*

When the supporting system is a piping system, the supported pipe (branch) can be decoupled from the supporting pipe (run) when the ratio of the run piping nominal pipe size to branch pipe nominal pipe size is greater than or equal to three to one. Decoupling can also be done when the moment of inertia of the branch pipe is less than or equal to 4 percent of the moment of inertia of the run pipe.

During the analysis of the branch piping, resulting values of tee anchor reactions are checked against the capabilities of the tee.

The seismic inertia effects of equipment and piping that provide support to supported (branch) piping systems are considered when significant. When the frequency of the supporting equipment is less than 33 hertz, then either a coupled dynamic model of the piping and equipment is used, or the amplified response spectra at the equipment connection point is used with a decoupled model of the supported piping. When supported piping is supported by larger piping, one of the following methods is used:

- *A coupled dynamic model of the supported piping and the supporting piping*
- *Amplified response spectra at the connection point to the supporting piping with a decoupled model of the supported piping*]*

3.7.3.8.2.2 Small-Diameter Auxiliary Piping

[*This subsection deals with ASME Code Class 1 piping equal to or less than 1-inch nominal pipe size and ASME Class 2 and 3 piping with nominal pipe sizes less than or equal to 2 inches. This includes instrumentation tubing. These piping systems may be supported by equipment or primary loop piping or other auxiliary piping or both. The response spectra or equivalent static load methodology is used. One of the following methods may be used for these systems:*

- *Same method as described in subsection 3.7.3.8.2.1*
- *Equivalent static analysis based on appropriate load factors applied to the response spectra acceleration values*]*

Subsections 3.9.3 and 3.9.8.2 discuss the final design and as built reconciliation of small bore piping.

3.7.3.8.3 Piping Systems on Modules

Many portions of the systems for the AP1000 are assembled as modules offsite and shipped to the plant as completed units. This method of construction does not result in any unique requirements for the analysis of these structures, systems, or components. Existing industry standards and regulatory requirements and guidelines are appropriate for the evaluation of structures, systems, and components included in modules.

The modules are constructed using a structural steel framework to support the equipment, pipe, and pipe supports in the module. The structural steel framework is designed as part of the building structure according to the criteria given in Subsection 3.8.4.

*NRC Staff approval is required prior to implementing a change in this information.

One exception is the pressurizer and safety relief valve module, which is attached to the top of the pressurizer. For this module the structures and piping arrangements support valves off the pressurizer and not the building structure. The structural steel frame is designed as a component support according to ASME Code, Section III, Subsection NF. [*Piping in modules is routed and analyzed in the same manner as in a plant not employing modules. Piping is analyzed from anchor point to anchor point, which are not necessarily at the boundaries of the module.*]* This is consistent with the manner in which room walls are treated in a nonmodule plant.

[*The supported piping or component may be decoupled from the seismic analysis of the structural frame based on the following criteria. The mass ratio, Rm, and the frequency ratio, Rf, are defined as follows:*

- *Rm = mass of supported component or piping/mass of supporting structural frame*
- *Rf = frequency of the component or piping/frequency of the structural frame*

Decoupling may be done when:

- *Rm < 0.01, for any Rf, or*
- *Rm $\geq$ 0.01 and $\leq$ 0.10, if Rf $\leq$ 0.8 or if Rf is $\geq$ 1.25.*

In addition, supported piping may be decoupled if analysis shows that the effect on the structural frame is small, that is, when the change in response is less than 10 percent. When piping or components are decoupled from the analysis of the frame, the contributory mass of the piping and components is included as a rigid mass in the model of the structural frame.]*

When piping or components are decoupled from the analysis of the frame using the preceding criteria, the effect of the frame is accounted for in the analysis of the decoupled components or piping. Either an amplified response spectra or a coupled model is used. The amplified response spectra are obtained from the time history safe shutdown earthquake analysis of the frame. The coupled model consists of a simplified mass and stiffness model of the frame connected to the seismic model of the components or piping.

Alternative criteria may be applied to simple frames that behave as pipe support miscellaneous steel. Decoupling may be done when the deflection of the frame due to dynamic combined faulted loading is less than or equal to 1/8 inch. These deflections are defined with respect to the structure to which the structural frame is attached. The stiffness of the intervening elements between the frame and the supported piping or component is considered as follows: Default rigid stiffness values are used for supports except that vendor stiffness values are used for snubbers and rigid gapped supports. The mass of the structural frame is evaluated as a self-weight excitation loading on the frame and the structures supporting the frame. The same approach is used for pipe support miscellaneous steel, as described in Subsection 3.9.3.4.

When the supported components or piping cannot be decoupled, they are included in the analysis model of the structural frame. The interaction between the piping and the frame is incorporated by including the appropriate stiffness and mass properties of the components, piping, and frame in the coupled model.

[*3.7.3.8.4 Piping Systems with Gapped Supports*

This subsection describes the analysis methods for piping systems with rigid gapped supports. These supports may be used to minimize the number of pipe support snubbers and the corresponding inservice testing and maintenance activities.

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The analysis consists of an iterative response spectra analysis of the piping and support system. Iterations are performed to establish calculated piping displacements that are compatible with the stiffness and gap of the rigid gapped supports. The results of the computer program GAPPIPE, which uses this methodology, are supported with test data (Reference 13).

The method implemented in GAPPIPE to analyze piping systems supported by rigid gapped supports is based on the equivalent linearization technique. GAPPIPE analysis is performed whenever snubber supports are replaced by rigid gapped supports.

The basis of the concept is to find an equivalent linear spring with a response-dependent stiffness for each nonlinear rigid gapped support, or limit stop, in the mathematical model of the piping system. The equivalent linearized stiffness minimizes the mean difference in force in the support between the equivalent spring and the corresponding original gapped support. The mean difference is estimated by an averaging process in the time domain, that is, across the response duration, using the concept of random vibration. Details of the design and analysis methods and modeling assumptions are described in Reference 12.]*

3.7.3.9 Combination of Support Responses

This subsection describes alternative methods for combining the responses from the individual support or attachment points that connect the supported system or subsystem to the supporting system or subsystem. There are two aspects to the responses from the support or attachment points: seismic anchor motions and envelope or multiple-input response spectra methodology.

Seismic Anchor Motions – The response due to differential seismic anchor motions is calculated using static analysis (without including a dynamic load factor). In this analysis, the static model is identical to the static portion of the dynamic model used to compute the seismic response due to inertial loading. In particular, the structural system supports in the static model are identical to those in the dynamic model.

[*The effect of relative seismic anchor displacements is obtained either by using the worst combination of the peak displacements or by proper representation of the relative phasing characteristics associated with different support inputs. For components supported by a single concrete building (coupled shield and auxiliary buildings, or containment internal structures), the seismic motions at all elevations above the basemat are taken to be in phase. When the component supports are in the same structure, the relative seismic anchor motions are small and the effects are neglected. This is applicable to building structures and to those supplemental steel frames that are rigid in comparison to the components. Supplemental steel frames that are flexible can have significant seismic anchor motions which are considered. When the components supports are in different structures, the relative seismic anchor motion between the structures is taken to be out-of-phase and the effects are considered. The results of the modal spectra analysis (multiple input or envelope) are combined with the results from seismic anchor motion by the absolute sum method or the SRSS method, as described in Tables 3.9-5 and 3.9-6.*]*

Response Spectra Methods – The envelope broadened uniform-input response spectra can lead to excessive conservatism and unnecessary pipe supports. The peak shifting method and independent support motion spectra method are used to avoid unnecessary conservatism.

Seismic Response Spectra Peak Shifting

The peak shifting method may be used in place of the broadened spectra method, as described below.

Determine the natural frequencies $(f_e)_n$ of the system to be qualified in the broadened range of the maximum spectrum acceleration peak.

*NRC Staff approval is required prior to implementing a change in this information.

If no equipment or piping system natural frequencies exist in the ±15 percent interval associated with the maximum spectrum acceleration peak, then the interval associated with the next highest spectrum acceleration peak is selected and used in the following procedure.

Consider all N natural frequencies in the interval

$$f_j - 0.15f_j \leq (f_e)_n \leq f_j + 0.15f_j$$

where:

f_j = the frequency of maximum acceleration in the envelope spectra

n = 1 to N

The system is then evaluated by performing N + 3 separate analyses using the envelope unbroadened floor design response spectrum and the envelope unbroadened spectrum modified by shifting the frequencies associated with each of the spectral values by a factor of +0.15; -0.15; and

$$\frac{(f_e)_n - f_j}{f_j}$$

where:

n = 1 to N

The results of these separate seismic analyses are then enveloped to obtain the final result desired (e.g., stress, support loads, acceleration, etc.) at any given point in the system. If three different floor response spectrum curves are used to define the response in the two horizontal and the vertical directions, then the shifting of the spectral values as defined above may be applied independently to these three response spectrum curves.

Independent Support Response Spectrum Methods

The use of multiple-input response spectra accounts for the phasing and interdependence characteristics of the various support points. The following alternative methods are used for the AP1000 plant. These are based on the guidelines provided by the "Pressure Vessel Research Committee Technical Committee on Piping Systems" (Reference 14).

[*Envelope Uniform Response Spectra - Method A - The seismic response spectrum that envelopes the supports is used in place of the spectra at each support in the envelope uniform response spectra. Also, the contribution from the seismic anchor motion of the support points is assumed to be in phase and is added algebraically as follows:*

$$q_i = d_i \sum_{j=1}^{N} P_{ij}$$

where:

q_i = *combined displacement response in the normal coordinate for mode i*

d_i = *maximum value of* d_{ij}

*NRC Staff approval is required prior to implementing a change in this information.

d_{ij} = *displacement spectral value for mode i associated with support "j"*

P_{ij} = *participation factor for mode i associated with support j*

N = *number of support points*

Enveloped response spectra are developed as the seismic input in three perpendicular directions of the piping coordinate system to include the spectra at the floor elevations of the attachment points and the piping module or equipment if applicable. The mode shapes and frequencies below the cut-off frequency are calculated in the response spectrum analysis. The modal participation factors in each direction of the earthquake motion and the spectral accelerations for each significant mode are calculated. Based on the calculated mode shapes, participation factors, and spectral accelerations of individual modes, the modal inertia response forces, moments, displacements, and accelerations are calculated. For a given direction, these modal inertia responses are combined based on consideration of closely spaced modes and high frequency modes to obtain the resultant forces, moments, displacements, accelerations, and support loads. The total seismic responses are combined by square-root-sum-of-the-squares method for all three earthquake directions.

Independent Support Motion - Method B - When there are more than one supporting structure, the independent support motion (ISM) method for seismic response spectra may be used.

Each support group is considered to be in a random-phase relationship to the other support groups. The responses caused by each support group are combined by the square-root-sum-of-the-square method. The displacement response in the modal coordinate becomes:

$$q_i = \left[\sum_{j=1}^{N} \left(P_{ij}\, d_{ij} \right)^2 \right]^{1/2}$$

A support group is defined by supports that have the same time-history input. This usually means all supports located on the same floor (or portions of a floor) of a structure.]*

3.7.3.10 Vertical Static Factors

Constant static factors can be used in some cases for the design of seismic Category I subsystems and equipment. The criteria for using this method are presented in Subsection 3.7.3.5.

3.7.3.11 Torsional Effects of Eccentric Masses

[*The methods used to account for the torsional effects of valves and other eccentric masses (for example, valve operators) in the seismic subsystem analyses are as follows:*

- *When valves and other eccentric masses are considered rigid, the mass of the operator and valve body or other eccentric mass are located at their respective center of gravity. The eccentric components (that is, yoke, valve body) are modeled as rigid members.*

- *When valves and other eccentric masses are not considered rigid, the dynamic models are simulated by the lumped masses in discrete locations (that is, center of gravity of valve body and valve operator), coupled by elastic members with properties of the eccentric components.*]*

*NRC Staff approval is required prior to implementing a change in this information.

3.7.3.12 Seismic Category I Buried Piping Systems and Tunnels

[*There are no seismic Category I buried piping systems and tunnels in the AP1000 design.*]*

3.7.3.13 Interaction of Other Systems with Seismic Category I Systems

The safety functions of seismic Category I structures, systems, and components are protected from interaction with nonseismic structures, systems, and components; or their interaction is evaluated. The safety-related systems and components required for safe shutdown are described in Section 7.4. This equipment is located in selected areas of the auxiliary building and inside containment. The primary means of protecting safety-related structures, systems, and components from adverse seismic interactions are discussed in the following paragraphs in the order of preference.

- Separation – separation with the use of physical barriers
- Segregation – routing away from location of seismic Category I systems, structures, and components
- Impact Evaluation – contact with seismic Category I systems, structures, and components may occur, and there is insufficient energy in the impact to cause loss of safety function
- Support as seismic Category II

[*Interaction of connected systems with seismic Category I piping is considered by including the other piping in the analysis of the seismic Category I system.*]* Interaction of piping systems that are adjacent to Category I structures, systems, and components is also considered. This is discussed in Subsection 3.7.3.13.4.

The containment and each room outside containment containing safety-related systems or equipment, as identified in Table 3.7.3-1, are reviewed for potential adverse seismic interactions to demonstrate that systems, structures, and components are not prevented from performing their required safe shutdown functions. In addition, the review identifies the protection features required to mitigate the consequences of seismic interaction in an area that contains safety-related equipment.

The evaluation steps to address seismic interaction taken for each room or building area containing seismic Category I systems, structures, and components are:

1. Define targets susceptible to damage (sensitive targets);
   Sensitive targets are those seismic Category I components for which adverse spatial interaction can result in loss of safety function.
2. Define sources which can potentially interact in an adverse manner with the target.
3. If possible, assure adequate free space to eliminate the possibility of seismically-induced damaging impacts for the sensitive targets.
4. Assess impact effects (interaction) when adequate free space is not present.
5. Correct adverse seismic interaction conditions.

The three-dimensional computer model and composites developed for the nuclear island are used during the design process of the systems and components in the nuclear island, to aid in evaluating and documenting the review for seismic interactions. This review is performed using the design criteria and guidelines described in Subsections 3.7.3.13.1 through 3.7.3.13.4.

*NRC Staff approval is required prior to implementing a change in this information.

The seismic interaction review is discussed in Subsection 3.7.5.3. This review is performed in parallel with the seismic margin evaluation. The review is based on as-procured data, as well as the as-constructed condition.

3.7.3.13.1 Separation and Segregation

Separation – The general plant arrangement provides physical separation between the seismic Category I and nonseismic structures, systems, and components to the maximum extent practicable in the nuclear island. The objective is to assist in the preclusion of a potential adverse interaction if the nonseismic structures, systems and components were to fail during a seismic event. Whenever possible, nonseismic pipe, electrical raceway, or ductwork is not routed above or adjacent to safety-related equipment, pipe, electrical raceway, or ductwork, thereby eliminating the possibility of seismic interaction.

Workstations and other equipment in the Main Control Room are separated from piping. Further, as stated in Subsection 3.2.1.1.2, structures, systems, and components that are located overhead in the Main Control Room are supported as seismic Category II.

Segregation – Where separation by physical means cannot be accomplished and it becomes necessary to locate or route nonseismic structures, systems, and components in or through safety-related areas, the nonseismic structures, systems and components are segregated from the seismic Category I items to the extent practicable.

Nonseismic cabinets are separated or segregated from seismic Category I cabinets. Also, if a cabinet is a source or a target, the cabinet doors must be secured by latches or fasteners to assure they do not open during a seismic event.

3.7.3.13.2 Impact Analysis

Adverse spatial interaction (i.e., loss of structural integrity or function effecting safety) can potentially occur when two items are in close proximity. Adverse spatial interaction can result from contact or impact from overturning. Seismic Category I systems, structures, and components that are sensitive to seismic interaction are identified as potential targets. Sources are structures or components that can have adverse spatial interaction with the seismic Category I systems, structures, and components. Identification and evaluation of spatial interactions includes the following considerations:

- Proximity of the source to the target. That is, the location of the source within the impact evaluation zone (shown in Figure 3.7.3-1)

  If a source is outside the impact evaluation zone, and does not enter this zone if overturning occurs, no adverse spatial interaction can occur with the identified target. If the source is within the impact evaluation zone and the supports of the source fail, the source could free fall, potentially impacting the target.

- Robustness of target

  If a target has significant structural integrity, and its function is not an issue, adverse spatial interaction could not occur with the identified source.

- Energy of impact

  The energy of the source impacting the target may be so low as not to cause adverse spatial interaction with the target.

A specific nonseismic structure, system, or component identified as a source to a specific safety-related component can be acceptable without being supported as seismic Category II, if an analysis demonstrates that the weight and configuration of the source, relative to the target, and the trajectory of the source are such that the interaction would not cause unacceptable damage to the target. For example, a nonseismic instrument tube routed above a seismic Category I electrical cable tray would not pose a hazard and would be acceptable.

Nonseismic equipment can overturn as a result of a safe shutdown earthquake. The trajectory of its fall is evaluated to determine if it poses a potential impact hazard to a safety-related structure, system, or component. If it poses a hazard, the equipment is relocated, or it is supported as described in Subsection 3.7.3.13.3.

Nonseismic walls, platforms, stairs, ladders, grating, handrail installations, or other structures next to safety-related structures, systems, and components are evaluated to determine if their failure is credible.

Should a nonseismic structure, system, or component be capable of being dislodged from its supports, the trajectory of its fall is evaluated for potential adverse impacts. If these present a hazard, the structure, system or component is relocated or supported as described in Subsections 3.7.3.13.3 and 3.7.3.13.4. Impact is assumed for sources within an impact evaluation zone around the safety-related equipment. The impact evaluation zone is defined as the envelope around the target for which a source, if located outside of the envelope, would not impact the target during a safe shutdown earthquake in the event the supports of the source were to fail and allow the source to fall. The impact evaluation zone is defined by the volume extending 6 feet horizontally from the perimeter of the seismic Category I object up to a height of 35 feet. The impact evaluation zone above 35 feet is defined by a 10-degree cone radiating vertically from the foot of the object, projected from its perimeter. This definition of the impact evaluation zone is illustrated in Figure 3.7.3-1. The impact evaluation zone need not extend beyond seismic Category I structures such as walls or floor slabs.

Using seismic experience data, the following seismic Category I equipment (potential targets) are not sensitive to piping, HVAC ducts, and cable tray interaction because they are robust to these types of impact:

- Tanks, "heavy" equipment (for example, heat exchangers)
- Mechanical or electrical penetrations
- Heating, ventilation, and air conditioning (HVAC)
- Adjacent piping
- Conduits
- Cable trays
- Structures

3.7.3.13.3 Seismic Category II Supports

Where the preceding approaches of separation, segregation, or impact analysis cannot prevent unacceptable interaction, the source is classified and supported as seismic Category II. The seismic Category II designation provides confidence that these nonseismic structures, systems, and components can withstand the forces of a safe shutdown earthquake in addition to the loading imparted on the seismic Category II supports due to failure of the remaining nonseismically supported portions. This includes nozzle loads from the nonseismic piping. Design methods and stress criteria for systems, structures, and components classified as seismic Category II are the same as for seismic Category I systems, structures, and components, except for piping which is described in Subsection 3.7.3.13.4.2. However, the functionality of these seismic Category II sources does not have to be maintained following a safe shutdown earthquake.

HVAC duct and/or cable trays within the impact evaluation zone are seismically supported using the criteria given in Appendices 3F and 3A for seismic Category I assuring that the HVAC and cable tray segments identified as a source will not fall or adversely impact the sensitive target. Adequate free space between the source and target is assured using the load combination that includes the safe shutdown earthquake. The seismic displacement of the HVAC duct and/or cable tray is 6 inches or the calculated displacement.

Nonseismic equipment identified as a source within the impact evaluation zone is supported as seismic Category II. Support seismic loads include seismic inertia loads of the equipment determined as described in Subsection 3.7.3.5 and nozzle loads from attached piping determined as described in Subsection 3.7.3.13.4.2. Adequate free space is assessed considering a 6-inch deflection envelope for equipment identified as a source, or calculated deflections obtained using the safe shutdown earthquake load combination and elastic analysis.

[*3.7.3.13.4 Interaction of Piping with Seismic Category I Piping Systems, Structures, and Components*

This subsection describes the design methods for piping to prevent adverse spatial interactions.

3.7.3.13.4.1 Seismic Category I Piping

The safe shutdown earthquake piping displacements obtained for the seismic Category I piping are used for the evaluation of seismic interaction with sensitive equipment. Adequate free space between a source and a target is checked adding absolutely the piping safe shutdown earthquake deflection and the safe shutdown earthquake target deflection along with the other loads (e.g., dead weight, thermal) that are in the appropriate design criteria load combinations. Sensitive equipment for piping as the source is seismic Category I equipment shown in Table 3.7.3-2 along with the portion that must be protected ("zone of protection"). Supports may be added to limit seismic movement to eliminate potential adverse interaction.

3.7.3.13.4.2 Seismic Category II Piping

This subsection describes the methods and criteria for piping that is connected to seismic Category I piping. Interaction of seismic Category I piping and nonseismic Category I piping connected to it is achieved by incorporating into the analysis of the seismic Category I system a length of pipe that represents the actual dynamic behavior of the complete run of the nonseismic Category I system. The length considered is classified as seismic Category II and extends to the interface anchor or rigid support as described below.

The seismic Category II portion of the line, up to the interface anchor or interface rigid support (last seismic support), is analyzed according to Equation 9 of ASME Code, Section III, Class 3, with a stress limit equal to the smaller of 4.5 S_h and 3.0 S_y. In either case, the nonseismic piping is isolated from the seismic Category I piping by anchors or seismic supports. The anchor or seismic Category II supports are designed for loads from the nonseismic piping. This includes three plastic moment components (M_{p1}, M_{p2}, or M_{p3}) in each of three local coordinate directions. The responses to the three moments are evaluated independently. The seismic Category II portion of the line is analyzed by the response spectrum or equivalent static load method for safe shutdown earthquake.

Single Interface Anchor

The seismic Category II piping may be terminated at a single interface anchor (six-way). This anchor and the supports on the seismic Category II piping are evaluated for safe shutdown earthquake loadings using the rules of ASME III Subsection NF. If the anchor is an equipment nozzle, then the equipment load path through the equipment supports are evaluated to the same acceptance criteria as seismic Category I equipment.

*NRC Staff approval is required prior to implementing a change in this information.

Anchor Followed by a Series of Seismic Supports

The seismic Category II piping may be terminated at the last seismic support which follows a six-way anchor on the seismic Category II piping. This last seismic support and the supports on the seismic Category II piping are evaluated for safe shutdown earthquake loadings using the rules of ASME III Subsection NF. From the anchor to the last seismic support, the response to the plastic moments (M_{p1}, M_{p2}, or M_{p3}) is combined with the responses to seismic anchor motions and equivalent static seismic inertia of the piping system by the absolute sum method. The responses to these moments are evaluated independently. The support and anchor loads due to the plastic moments (M_{p1}, M_{p2}, or M_{p3}) of the seismically analyzed and supported section can be reduced if the elbow/bend resultant moments have exceeded the plastic limit moments of the elbow/bend. The value of the reduction factor RF is as follows:

RF	=	*Multiplier used to reduce the interface anchor and support loads*
RF	=	*< I, (if RF > I, no reduction is applicable)*
RF	=	M_L/M_a
M_a	=	*Resultant moment at elbow/bend. Use maximum value if several elbows/bends are within seismically supported region.*
M_L	=	$0.8h^{0.6}\ D^2t\ Sy$ *for* $h < 1.45$
M_L	=	$D^2t\ Sy$ *for* $h > 1.45$
h	=	*Flexibility characteristic of elbow/bend*
D	=	*Outside diameter of elbow/bend*
t	=	*Thickness of elbow/bend*
R	=	*Bend radius of elbow/bend*

Rigid Region

The seismic Category II piping may be terminated at the last seismic support of a rigidly supported region of the piping system. The rigid region is typically defined as either four bi-lateral supports around an elbow or six bilateral supports around a tee. The structural behavior of the rigid region is similar to that of a six-way anchor. The frequency of the piping system in the rigid region is greater than or equal to 33 hertz. This last seismic support in the rigid region and the supports on the seismic Category II piping are evaluated for safe shutdown earthquake loadings using the rules of ASME III Subsection NF.

3.7.3.13.4.3 Nonseismic Piping

Nonseismic piping within the impact evaluation zone is seismically supported, thereby ensuring that the pipe segment identified as a source will not fall or adversely impact the sensitive target (Table 3.7-2). This situation is shown in Figure 3.7.3-2, and the seismic supported piping criteria described below:

> *Supports within the impact evaluation zone, plus one transverse support in each transverse direction beyond the impact evaluation zone, are classified as seismic Category II and are evaluated for the safe shutdown earthquake loading using the rules of ASME III, Subsection NF.*

*NRC Staff approval is required prior to implementing a change in this information.

- *Piping within the impact evaluation zone plus one transverse support in each transverse direction are evaluated to Equation 9 of ASME Code, Section III, Class 3, with a stress limit equal to the smaller of 4.5 S_h and 3.0 S_y. Outside the impact evaluation zone, the nonseismic piping meets ASME/ANSI B31.1 requirements.*

- *The nonseismic piping and seismic Category II supports are designed for loads from the nonseismic piping beyond the impact evaluation zone. This includes three plastic moment components (M_{p1}, M_{p2}, or M_{p3}) in each of three local coordinate directions applied at the first and last seismic Category II support. The responses to the three moments are evaluated independently. The response from the moments applied at the first seismic*

- *Category II support is combined with the response from the moments applied at the last seismic Category II support and with the responses to seismic anchor motions and equivalent static seismic inertia of the piping system by the absolute sum method. The support and anchor loads due to the plastic moments (M_{p1}, M_{p2}, or M_{p3}) of the seismically analyzed and supported section can be reduced if the elbow/bend resultant moments have exceeded the plastic limit moments of the elbow/bend. The value of the reduction factor RF is the same as the value for connected seismic Category II piping described above.*

- *The piping segment identified as the source has at least one effective axial support.*

- *Adequate free space between a source and a target is checked adding absolutely the piping safe shutdown earthquake deflections (defined following seismic Category II piping analysis methodology) and the safe shutdown earthquake target deflection. Also included are the displacements associated with the appropriate load cases.*

- *When the anchor is an equipment nozzle, the equipment is supported as seismic Category II as described in subsection 3.7.3.13.3.*]*

3.7.3.14 Seismic Analyses for Reactor Internals

See Subsection 3.9.2 for the dynamic analyses of reactor internals.

3.7.3.15 Analysis Procedure for Damping

Damping values used in the seismic analyses of subsystems are presented in Subsection 3.7.1.3. Safe shutdown earthquake damping values used for different types of analysis are provided in Table 3.7.1-1. For subsystems that are composed of different material types, the composite modal damping approach with the weighted stiffness method is used to determine the composite modal damping value. Alternately, the minimum damping value may be used for these systems. [*Composite modal damping for coupled building and piping systems is used for piping systems that are coupled to the primary coolant loop system and the interior concrete building. Composite modal damping is used for piping systems that are coupled to flexible equipment or flexible valves. Piping systems analyzed by the uniform envelope response spectra method with rigid valves can be evaluated with 5 percent damping. Five percent damping is not used in piping systems that are susceptible to stress corrosion cracking.*]*

For the time history dynamic analysis and independent support motion response spectra analysis of piping systems, 4 percent, 3 percent, and 2 percent damping values are used as described in Table 3.7.1-1.

When piping systems and nonsimple module steel frames (simple frames are described in Subsection 3.7.3.8.3) are in a single coupled model, composite damping, as described in Subsection 3.7.1.3 is used.

*NRC Staff approval is required prior to implementing a change in this information.

3.7.3.16 Analysis of Seismic Category I Tanks

This subsection describes the seismic analyses for the large, atmospheric seismic Category I pools and tanks. These are reinforced concrete structures with stainless steel liners or with structural modules, as discussed in Subsections 3.8.3 and 3.8.4. They include the spent fuel pit in the auxiliary building, the in-containment refueling water storage tank, and the passive containment cooling water tank incorporated into the shield building roof. There are no other seismic Category I tanks.

The seismic analyses of the tank consider the impulsive and convective forces of the water as well as the flexibility of the walls. For the spent fuel pit, cask loading pit, cask washdown pit and fuel transfer canal, the impulsive loads are calculated by considering a portion of the water mass responding with the concrete walls. The impulsive forces are calculated by conventional methods for rigid tanks. The passive containment cooling water tank is analyzed using methods described in Reference 15 for toroidal tanks. It is also analyzed by finite element methods. The in-containment refueling water storage tank is irregular in plan and is analyzed by finite element methods.

3.7.3.17 Time History Analysis of Piping Systems

[*The time history dynamic analysis is an alternate seismic analysis method for response spectrum analysis when time history seismic input is used. This method is also used for dynamic analyses of piping systems subjected to time history hydraulic transient loadings or forcing functions induced by postulated pipe breaks. The modal superposition method is used to solve the equations of motion. The computer programs used are GAPPIPE, PIPESTRESS, ANSYS, and WECAN.*

The modal superposition method is based on the equations of motion which can be decoupled as long as the piping system is within its elastic limit. The modal responses are obtained from integrating the decoupled equations. The total responses are obtained by the algebraic sum of the individual responses of the individual modes at each time step. The cutoff frequency is selected based on the frequency content of the input forcing function and the highest significant frequency of the piping system. The integration time step is no larger than 10 percent of the period of the cutoff frequency.

For dynamic analysis, including seismic analysis at a hard rock site, three separate analyses are performed for each loading case to account for uncertainties. The three analyses correspond to three different time scales: normal time, time expanded by 15 percent, and time compressed by 15 percent. For time history analysis of piping system models that include a dynamic model of the supporting concrete building either the building stiffness is varied by + and - 30 percent, or the time scale is shifted by + and - 15 percent. Alternately, when uniform enveloping time history analysis is performed, modeling uncertainties are accounted for by the spreading that is included in the broadened response spectra.

For time history analysis using the PIPESTRESS program, the response from the high frequency modes above the cutoff frequency is calculated based on the static response to the left-out-forces. This response is combined with the response from the low frequency modes by algebraic sum at each time step. Composite modal damping is used with PIPESTRESS program. The damping of the individual components is as listed in Table 3.7.1-1.

Alternately, for time history analysis using the PIPESTRESS, GAPPIPE, ANSYS, or WECAN programs, the number of modes used in the modal analysis is chosen so that the results of the dynamic analysis based on the chosen number of modes are within 10 percent of the results of the dynamic analysis based on the next higher number of modes used. The number of modes analyzed is selected to account for the principal vibration modes of the piping system. The modes are combined by algebraic sum. Composite modal damping is used with the ANSYS or WECAN programs. The damping of the individual components is as listed in Table 3.7.1-1.]*

*NRC Staff approval is required prior to implementing a change in this information.

3.7.4 Seismic Instrumentation

3.7.4.1 Comparison with Regulatory Guide 1.12

Compliance with Regulatory Guide 1.12 is discussed in this section and in Subsection 1.9.1.

Administrative procedures define the maintenance and repair of the seismic instrumentation to keep the maximum number of instruments in-service during plant operation and shutdown in accordance with Regulatory Guide 1.12.

3.7.4.1.1 Safety Design Basis

The seismic instrumentation serves no safety-related function and therefore has no nuclear safety design basis.

3.7.4.1.2 Power Generation Design Basis

The seismic instrumentation is designed to provide the following:

- Collection of seismic data in digital format
- Analysis of seismic data after a seismic event
- Operator notification that a seismic event exceeding a preset value has occurred
- Operator notification (after analysis of data) that a predetermined cumulative absolute velocity value has been exceeded

3.7.4.2 Location and Description of Instrumentation

The following instrumentation and associated equipment are used to measure plant response to earthquake motion.Four triaxial acceleration sensor units, located as stated in Subsection 3.7.4.2.1, are connected to a time-history analyzer. The time-history analyzer recording and playback system is located in a panel in the nuclear island in a room near the main control room. Seismic event data from these sensors are recorded on a solid-state digital recording system at 200 samples per second per data channel.

This solid-state recording and analysis system has internal batteries and a charger to prevent the loss of data during a power outage, and to allow data collection and analysis in a seismic event during which the power fails. Normally 120 volt alternating current power is supplied from the non-Class 1E dc and uninterruptible power supply system. The system uses triaxial acceleration sensor input signals to initiate the time-history analyzer recording and main control room alarms. The system initiation value is adjustable from 0.002g to 0.02g.

The time-history analyzer starts recording triaxial acceleration data from each of the triaxial acceleration sensors after the initiation value has been exceeded. Pre-event recording time is adjustable from 1.2 to 15.0 seconds, and will be set to record at least 3 seconds of pre-event signal. Post-event run time is adjustable from 10 to 90 seconds. A minimum of 25 minutes of continuous recording is provided. Each recording channel has an associated timing mark record with 2 marks per second, with an accuracy of about 0.02 percent.

The instrumentation components are qualified to IEEE 344-1987 (Reference 16).

The sensor installation anchors are rigid so that the vibratory transmissibility over the design spectra frequency range is essentially unity.

3.7.4.2.1 Triaxial Acceleration Sensors

Each sensor unit contains three accelerometers mounted in a mutually orthogonal array mounted with one horizontal axis parallel to the major axis assumed in the seismic analysis. The triaxial acceleration sensors have a dynamic range of 1000 to 1 (0.001 to 1.0g) and a frequency range of 0.2 to 50 hertz.

One sensor unit will be located in the free field, as discussed below. The AP1000 seismic monitoring system will provide for signal input from the free field sensor.

A second sensor unit is located on the nuclear island basemat in the spare battery charger room at elevation 66′-6″ near column lines 9 and L.

A third sensor unit is located on the shield building structure at elevation 266′ near column lines 4-1 and K.

The fourth sensor unit is located on the containment internal structure on the east wall of the east steam generator compartment just above the operating floor at elevation 138′ close to column lines 6 and K.

Seismic instrumentation is not located on equipment, piping, or supports since experience has shown that data obtained at these locations are obscured by vibratory motion associated with normal plant operation.

A free-field sensor will be located and installed to record the ground surface motion representative of the site. It will be located such that the effects associated with surface features, buildings, and components on the recorded ground motion will be insignificant. The trigger value is initially set at 0.01 g.

3.7.4.2.2 Time-History Analyzer

The time-history analyzer receives input from the triaxial acceleration sensors and, when activated as described in Subsection 3.7.4.3, begins recording the triaxial data from each triaxial acceleration sensor and initiates audio and visual alarms in the main control room.

This recorded data will be used to evaluate the seismic acceleration of the structure on which the triaxial acceleration sensors are mounted.

The time-history analyzer is a multichannel, digital recording system with the capability to automatically download the recorded acceleration data to a dedicated computer for data storage, playback, and analysis after a seismic event.

The time-history analyzer can compute cumulative absolute velocity (CAV) and the 5 percent of critical damping response spectrum for frequencies between 1 and 10 Hz. The operator may select the analysis of either CAV or the response spectrum. Analysis results are printed out on a dedicated graphics printer that is part of the system and is located in the same panel as the time-history analyzer.

3.7.4.3 Control Room Operator Notification

The time-history analyzer provides for initiation of audible and visual alarms in the main control room when predetermined seismic acceleration values sensed by any of the triaxial acceleration sensors are exceeded and when the system is activated to record a seismic event. In addition to alarming when the system is activated, the analyzer portion of the system will provide a second alarm if the predetermined cumulative absolute velocity value has been exceeded by any of the sensors. Alarms are annunciated in the main control room.

3.7.4.4 Comparison of Measured and Predicted Responses

The recorded seismic data is used by the combined license holder operations and engineering departments to evaluate the effects of the earthquake on the plant structures and equipment.

The criterion for initiating a plant shutdown following a seismic event will be exceedance of a specified response spectrum limit or a cumulative absolute velocity limit. The seismic instrumentation system is capable of computing the cumulative absolute velocity as described in EPRI Report NP-5930 (Reference 1) and EPRI Report TR-100082 (Reference 17).

Post-earthquake operating procedures utilize the guidance of EPRI Reports NP-5930, TR-100082, and NP-6695, as modified and endorsed by the NRC in Regulatory Guides 1.166 and 1.167. A response spectrum check up to 10 Hz will be based on the foundation instrument. The cumulative absolute velocity will be calculated based on the recorded motions at the free field instrument. If the operating basis earthquake ground motion is exceeded or significant plant damage occurs, the plant must be shutdown in an orderly manner.

In addition, the procedures address measurement of the post-seismic event gaps between the new fuel rack and walls of the new fuel storage pit, between the individual spent fuel racks, and from the spent fuel racks to the spent fuel pool walls, and provide for appropriate corrective actions to be taken if needed (such as repositioning the racks or analysis of the as-found condition).

3.7.4.5 Tests and Inspections

Periodic testing of the seismic instrumentation system is accomplished by the functional test feature included in the software of the time-history recording accelerograph. The system is modular and is capable of single-channel testing or single channel maintenance without disabling the remainder of the system.

Installation and acceptance testing of the triaxial acceleration sensors described in Subsection 3.7.4.2.1 is completed prior to initial startup. Installation and acceptance testing of the time-history analyzer described in Subsection 3.7.4.2.2 is completed prior to initial startup.

3.7.5 Combined License Information

3.7.5.1 Seismic Analysis of Dams

Dams whose failure could affect the site interface flood level are addressed in Subsection 3.7.2.12.

3.7.5.2 Post-Earthquake Procedures

Site-specific procedures for activities following an earthquake are addressed in Subsection 3.7.4.4.

3.7.5.3 Seismic Interaction Review

The seismic interaction review will be updated for as-built information. This review is performed in parallel with the seismic margin evaluation. The review is based on as-procured data, as well as the as-constructed condition. The as-built seismic interaction review is completed prior to fuel load.

3.7.5.4 Reconciliation of Seismic Analyses of Nuclear Island Structures

The seismic analyses described in Subsection 3.7.2 will be reconciled for detailed design changes, such as those due to as-procured or as-built changes in component mass, center of gravity, and support configuration based on as-procured equipment information. Deviations are acceptable based on an evaluation consistent with the methods and procedure of Section 3.7 provided the amplitude of the seismic floor response spectra, including the effect due to these deviations, does not exceed the design basis floor response spectra by more than 10 percent. This reconciliation will be completed prior to fuel load.

3.7.5.5 Free Field Acceleration Sensor

The location for the free-field acceleration sensor is addressed in Subsection 3.7.4.2.1.

3.7.6 References

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Table 3.7.1-1
Safe Shutdown Earthquake Damping Values

	Percent
Welded and friction-bolted steel structures and equipment	4
Bearing bolted structures and equipment	7
Prestressed concrete structures	5
Reinforced concrete structures	7
Concrete filled steel plate structures	5
[*Piping (for uniform envelope response spectra analysis)*	*5*
Piping (alternative for time history analysis and independent support motion response spectra analysis)	
Less than or equal to 12-inch diameter	*2*
Greater than 12-inch diameter	*3*
Primary coolant loop	*4*]*
Fuel assemblies	20
Control rod drive mechanisms	5
Full cable trays and related supports	10
Empty cable trays and related supports	7
Conduits and related supports	7
HVAC ductwork	7
HVAC welded ductwork	4
Cabinets and panels for electrical equipment	5
Equipment such as welded instrument racks and tanks	3

*NRC Staff approval is required prior to implementing a change in this information.

Table 3.7.1-2
Embedment Depth and Related
Dimensions of Category I Structures

Structure	Foundation Embedment Depth (ft)	Least Foundation Width (ft)	Structure Height (ft)
Shield Building	See Note	See Note	268.25
Steel Containment Vessel	See Note	See Note	215.33
Auxiliary Building	See Note	See Note	119.50

Note:

1. The seismic Category I structures are founded on a common basemat embedded 39.5 feet, [*with dimensions shown in Figure 3.7.1-14*.]*

*NRC Staff approval is required prior to implementing a change in this information.

Table 3.7.1-3
AP1000 Design Response Spectra
Amplification Factors for Control Points

HORIZONTAL					
Percent of Critical Damping	**Acceleration(1)**				**Displacement(1)**
	A (33 cps)	**B' (25 cps)(2)**	**B (9 cps)**	**C (2.5 cps)**	**D (0.25 cps)**
2.0	1.0	1.70	3.54	4.25	2.50
3.0	1.0	1.66	3.13	3.76	2.34
4.0	1.0	1.63	2.84	3.41	2.19
5.0	1.0	1.60	2.61	3.13	2.05
7.0	1.0	1.55	2.27	2.72	1.88
VERTICAL					
Percent of Critical Damping	**Acceleration(1)**				**Displacement(1)**
	A (33 cps)	**B' (25 cps)(2)**	**B (9 cps)**	**C (3.5 cps)**	**D (0.25 cps)**
2.0	1.0	1.70	3.54	4.05	1.67
3.0	1.0	1.66	3.13	3.58	1.56
4.0	1.0	1.63	2.84	3.25	1.46
5.0	1.0	1.60	2.61	2.98	1.37
7.0	1.0	1.55	2.27	2.59	1.25

Notes:
1. Maximum ground displacement is taken proportional to maximum ground acceleration, and is 36 inches for ground acceleration of 1.0 gravity.
2. The 5 percent damping amplification factor for control point B' is derived per discussion in Subsection 3.7.1.1. This 5 percent damping amplification factor equals 1.3 times the RG 1.60 response spectra at 25 hertz. The amplification factors at control point B' for other damping values are determined by increasing the RG 1.60 response spectra at 25 hertz by 30 percent.

Table 3.7.1-4 (Sheet 1 of 5)
Strain Compatible Soil Properties

Depth to Bottom of Layer (ft)	Thickness of Layer (ft)	Layer Number	Total Unit Weight (kcf)	Initial G (ksf)	Initial Vs (fps)	Final G (ksf)	Final Vs (fps)	Damping
Firm Rock								
0.0	–	–	–	–	–	–	–	–
5.0	5.0	1	0.15	57422	3500	57032	3499	0.015
10.0	5.0	2	0.15	57422	3500	56600	3486	0.016
15.0	5.0	3	0.15	57422	3500	55943	3465	0.017
20.0	5.0	4	0.15	57422	3500	55511	3452	0.018
25.0	5.0	5	0.15	56442	3500	55933	3465	0.016
30.0	5.0	6	0.15	56442	3500	55436	3450	0.017
33.5	3.5	7	0.15	57422	3500	56076	3470	0.015
39.5	6.0	8	0.15	57422	3500	55898	3464	0.015
45.0	5.5	9	0.15	57422	3500	55716	3458	0.016
50.0	5.0	10	0.15	57422	3500	55575	3454	0.016
60.0	10.0	11	0.15	56442	3500	55400	3449	0.017
80.0	20.0	12	0.15	56442	3500	54695	3427	0.019
100.0	20.0	13	0.15	56442	3500	53358	3384	0.021
120.0	20.0	14	0.15	56442	3500	52295	3351	0.023
Bedrock	–	–	0.15	300000	8000	298137	8000	0.02

Table 3.7.1-4 (Sheet 2 of 5)
Strain Compatible Soil Properties

Depth to Bottom of Layer (ft)	Thickness of Layer (ft)	Layer Number	Total Unit Weight (kcf)	Initial G (ksf)	Initial Vs (fps)	Final G (ksf)	Final Vs (fps)	Damping
Soft Rock								
0.0	–	–	–	–	–	–	–	–
5.0	5.0	1	0.15	27660	2429	27425	2426	0.016
10.0	5.0	2	0.15	29180	2495	28318	2466	0.018
15.0	5.0	3	0.15	30262	2541	28819	2487	0.020
20.0	5.0	4	0.15	30620	2556	28589	2477	0.023
25.0	5.0	5	0.15	30920	2568	29290	2508	0.019
30.0	5.0	6	0.15	31384	2588	29481	2516	0.021
33.5	3.5	7	0.15	31932	2610	30768	2570	0.017
39.5	6.0	8	0.15	32464	2632	31144	2586	0.018
45.0	5.5	9	0.15	33042	2655	31314	2593	0.019
50.0	5.0	10	0.15	33668	2680	31598	2604	0.020
60.0	10.0	11	0.15	34341	2707	31826	2614	0.021
80.0	20.0	12	0.15	35021	2733	31738	2610	0.024
100.0	20.0	13	0.15	35708	2760	31585	2604	0.026
120.0	20.0	14	0.15	36401	2787	31585	2604	0.028
Bedrock	–	–	0.15	300000	8000	298137	8000	0.020

Table 3.7.1-4 (Sheet 3 of 5)
Strain Compatible Soil Properties

Depth to Bottom of Layer (ft)	Thickness of Layer (ft)	Layer Number	Total Unit Weight (kcf)	Initial G (ksf)	Initial Vs (fps)	Final G (ksf)	Final Vs (fps)	Damping
Upper Bound Soft to Medium Soil								
0	–	–	–	–	–	–	–	–
5.0	5.0	1	0.11	6873	1414	6664	1397	0.018
10.0	5.0	2	0.11	9844	1692	9202	1641	0.023
15.0	5.0	3	0.11	13917	2012	12880	1942	0.024
20.0	5.0	4	0.11	14971	2087	13629	1997	0.027
25.0	5.0	5	0.11	15645	2133	14574	2065	0.022
30.0	5.0	6	0.11	16419	2186	15045	2099	0.024
33.5	3.5	7	0.11	17873	2280	16908	2225	0.019
39.5	6.0	8	0.11	19036	2353	17873	2287	0.020
45.0	5.5	9	0.11	20387	2435	18996	2358	0.021
50.0	5.0	10	0.11	21726	2514	20136	2428	0.021
60.0	10.0	11	0.11	23234	2600	21366	2501	0.022
80.0	20.0	12	0.11	24712	2681	22314	2556	0.024
100.0	20.0	13	0.11	26151	2758	23137	2602	0.026
120.0	20.0	14	0.11	27546	2831	24009	2651	0.027
Bedrock	–	–	0.15	300000	8000	298137	8000	0.020

Table 3.7.1-4 (Sheet 4 of 5)
Strain Compatible Soil Properties

Depth to Bottom of Layer (ft)	Thickness of Layer (ft)	Layer Number	Total Unit Weight (kcf)	Initial G (ksf)	Initial Vs (fps)	Final G (ksf)	Final Vs (fps)	Damping
Soft-to-Medium Soil								
0.0	–	–	–	–	–	–	–	–
5.0	5.0	1	0.11	3438	1000	3222	971	0.023
10.0	5.0	2	0.11	4923	1197	4355	1129	0.031
15.0	5.0	3	0.11	6960	1423	5987	1324	0.035
20.0	5.0	4	0.11	7487	1476	6161	1343	0.040
25.0	5.0	5	0.11	7824	1509	6699	1400	0.031
30.0	5.0	6	0.11	8211	1546	6891	1420	0.033
33.5	3.5	7	0.11	8938	1613	7872	1518	0.026
39.5	6.0	8	0.11	9520	1664	8317	1560	0.027
45.0	5.5	9	0.11	10195	1722	8834	1608	0.028
50.0	5.0	10	0.11	10864	1778	9347	1654	0.029
60.0	10.0	11	0.11	11618	1838	9818	1695	0.031
80.0	20.0	12	0.11	12357	1896	10031	1714	0.036
100.0	20.0	13	0.11	13077	1950	10201	1728	0.040
120.0	20.0	14	0.11	13774	2002	10512	1754	0.043
Bedrock	–		0.15	300000	8000	298137	8000	0.020

Table 3.7.1-4 (Sheet 5 of 5)
Strain Compatible Soil Properties

Depth to Bottom of Layer (ft)	Thickness of Layer (ft)	Layer Number	Total Unit Weight (kcf)	Initial G (ksf)	Initial Vs (fps)	Final G (ksf)	Final Vs (fps)	Damping
Soft Soil								
0.0	–	–	–	–	–	–	–	–
5.0	5.0	1	0.11	3438	1000	3222	971	0.023
10.0	5.0	2	0.11	3633	1028	3042	944	0.038
15.0	5.0	3	0.11	3865	1060	2974	933	0.047
20.0	5.0	4	0.11	3921	1068	2752	898	0.059
25.0	5.0	5	0.11	3955	1073	2922	925	0.049
30.0	5.0	6	0.11	3994	1078	2762	899	0.056
33.5	3.5	7	0.11	4065	1088	3022	941	0.046
39.5	6.0	8	0.11	4121	1095	2958	931	0.049
45.0	5.5	9	0.11	4183	1103	2896	921	0.053
50.0	5.0	10	0.11	4244	1111	2851	914	0.056
60.0	10.0	11	0.11	4310	1120	2774	901	0.062
80.0	20.0	12	0.11	4374	1128	2668	884	0.068
100.0	20.0	13	0.11	4434	1136	2691	888	0.069
120.0	20.0	14	0.11	4492	1143	2718	892	0.069
Bedrock	–	–	0.15	300000	8000	298137	8000	0.020

Tables 3.7.2-1–3.7.2-16 Not Used

Table 3.7.3-1 (Sheet 1 of 3)
Seismic Category I Equipment Outside Containment by Room Number

Room No.	Room Name	Equipment Description
12101	Division A battery room	Batteries
12102	Division C battery room 1	Batteries
12103	Spare battery room	Spare batteries
12104	Division B battery room 1	Batteries
12105	Division D battery room	Batteries
12113	Spare battery charger room	
12162	RNS pump room A	RNS pressure boundary
12163	RNS pump room B	RNS pressure boundary
12201	Division A dc equipment room	dc equipment
12202	Division C battery room 2	Batteries
12203	Division C dc equipment room	dc equipment
12204	Division B battery room 2	Batteries
12205	Division D dc equipment room	dc equipment
12207	Division B dc equipment room	dc equipment
12211	Corridor	Divisional cables
12212	Division B RCP trip switchgear room	RCP trip switchgear
12244	Lower annulus valve area	CVS/WLS containment isolation valves
12251	Demineralizer/filter access area	CVS/DWS isolation valves
12254	SFS penetration room	SFS containment isolation valve
12256	Containment isolation valve room	RNS containment isolation valves
12259	Pipe chase	RNS piping
12262	Piping/Valve room	RNS pressure boundary, SFS piping
12265	Waste monitor tank room C	SFS piping
12269	Pipe chase	RNS pressure boundary
12300	Corridor	Divisional cable, MCR load shed panel
12301	Division A I&C room	Divisional I&C
12302	Division C I&C room	Divisional I&C

Table 3.7.3-1 (Sheet 2 of 3)
Seismic Category I Equipment Outside Containment by Room Number

Room No.	Room Name	Equipment Description
12303	Remote shutdown room	Divisional cabling
12304	Division B I&C/penetration room	Divisional I&C/electrical penetrations
12305	Division D I&C/penetration room	Divisional I&C/electrical penetrations
12306	Valve/piping penetration room	CCS/CVS/DWS/FPS/SGS containment isolation valves
12311	Corridor	Divisional cabling
12312	Division C RCP trip switchgear room	RCP trip switchgear
12313	Division C I&C/penetration room	Divisional I&C/electrical penetrations
12321	Non-1E equipment/penetration room	Divisional cabling
12341	Middle annulus	Class 1E electrical penetrations Various mechanical piping penetrations
12351	Maintenance floor staging area	Divisional cabling (ceiling)
12352	Personnel hatch	Personnel airlock (interlocks)
12354	Middle annulus access room	PSS/SFS containment isolation valves
12362	RNS HX room	RNS pressure boundary
12365	Waste monitor tank room B	SFS piping
12400	Control room vestibule	Control room access
12401	Main control room	Dedicated safety panel VBS HVAC dampers VES isolation valves Lighting circuits Mounting for lighting fixtures
12404	Lower MSIV compartment B	SGS containment isolation valves, instrumentation and controls
12405	Lower VBS B and D equipment room	VWS/PXS/CAS containment isolation valves
12406	Lower MSIV compartment A	SGS containment isolation valves, instrumentation and controls
12412	Electrical penetration room Division A	Divisional electrical penetrations, MCR load shed panel

Table 3.7.3-1 (Sheet 3 of 3)
Seismic Category I Equipment Outside Containment by Room Number

Room No.	Room Name	Equipment Description
12421	Non 1E equipment/penetration room	Divisional cabling
12422	Reactor trip switchgear II	Reactor trip switchgear
12423	Reactor trip switchgear I	Reactor trip switchgear
12452	VFS penetration room	VFS containment isolation valves, divisional cabling
12454	VFS/SFS/PSS penetration room	SFS/PSS/VFS containment isolation valves, RNS pressure boundary
12462	Cask washdown pit	SFS piping
12504	Upper MSIV compartment B	SGS CIVs, instrumentation and controls
12506	Upper MSIV compartment A	SGS CIVs, instrumentation and controls
12541	Upper annulus	PCS piping and cabling PCS air baffle
12553	Personnel access area	Personnel airlock (interlocks)
12555	Operating deck staging area/VES air storage	VES high pressure air bottles
12651	VAS Equipment Room	VFS containment isolation valves
12562	Fuel handling area	Spent fuel storage racks
12701	PCS valve room	PCS isolation valves/instrumentation
12703	PCS water storage tank	PCS piping, level and temperature instrumentation

Table 3.7.3-2
Equipment Classified as Sensitive Targets for
Seismically Analyzed Piping, HVAC Ducting, Cable Trays

Component	Discussion	Zone of Protection
Seismic Category I Valve No Class 1E Electrical Equipment Not pressure sensitive	These are manual valves. The actuator must be protected from impact.	Valve body and actuator area
Seismic Category I Valve Class 1E Electrical Equipment Pressure sensitive	These valves have sensitive Class 1E equipment (e.g., Position indicators, limit switches, motor operator) or solenoid valves.	One support (acting in direction of impact) on each side of valve
Seismic Category I Dampers	The actuator must be protected along with any Class 1E equipment.	Within one support (acting in direction of impact) on each side of HVAC
Monitors	This includes: neutron detectors, radiation monitors, resistance temperature detectors, speed sensors, thermocouples, and transmitters.	Monitors and associated wiring
Sensitive Electrical Equipment Housed in Cabinets, Panels or Boards	This includes: relays, contractors, breakers, and switchgear.	Cabinets, panels, and boards housing sensitive devices
Class 1E exposed cables and wiring	Cables and wiring which are not housed in cable trays or conduits must be protected.	Exposed cables and wiring
Device or Instrument Tubing	Any device or tubing that could be damaged resulting in the loss of the pressure boundary of a safety class line.	Device or tubing
Penetrations	Rigid penetrations are considered robust. Floating penetrations with bellows are considered sensitive.	Floating penetration and associated bellows

Table 3.7-201 (Sheet 1 of 2)
Recommended Horizontal and Vertical FIRS
(Elevation –16 foot Horizon at Bottom of Nuclear Island Foundation)

FIRS Frequency (Hz)	Horizontal Sa(g)	Vertical Sa(g)
100	5.38E-02	5.38E-02
90	5.39E-02	5.39E-02
80	5.42E-02	5.42E-02
70	5.47E-02	5.47E-02
60	5.59E-02	5.59E-02
50	5.82E-02	5.82E-02
45	6.01E-02	6.01E-02
40	6.25E-02	6.25E-02
35	6.76E-02	6.76E-02
30	7.78E-02	7.78E-02
25	9.41E-02	9.41E-02
20	9.83E-02	9.83E-02
15	8.59E-02	8.59E-02
12.5	8.07E-02	8.07E-02
10	8.17E-02	8.17E-02
9	8.34E-02	8.34E-02
8	8.47E-02	8.47E-02
7	8.34E-02	8.34E-02
6	8.04E-02	8.04E-02
5	8.71E-02	8.70E-02
4	7.97E-02	7.96E-02
3	8.77E-02	7.51E-02
2.5	9.45E-02	6.76E-02
2	7.94E-02	5.64E-02
1.5	6.92E-02	4.87E-02
1.25	7.43E-02	5.20E-02
1	8.59E-02	5.98E-02
0.9	9.94E-02	6.89E-02
0.8	1.07E-01	7.42E-02
0.7	1.01E-01	6.97E-02
0.6	9.46E-02	6.49E-02
0.5	8.04E-02	5.48E-02
0.4	5.02E-02	3.40E-02
0.3	3.21E-02	2.16E-02
0.2	2.09E-02	1.40E-02
0.15	1.34E-02	8.93E-03

Table 3.7-201 (Sheet 2 of 2)
Recommended Horizontal and Vertical FIRS
(Elevation –16 foot Horizon at Bottom of Nuclear Island Foundation)

FIRS Frequency (Hz)	Horizontal Sa(g)	Vertical Sa(g)
0.125	9.69E-03	6.48E-03
0.1	5.83E-03	3.90E-03

Table 3.7-202
Total Static and Pseudo-Dynamic Loads (ksf)

Turbine Building	Multiplier of 1	Multiplier of 2
First Bay	-5.03	-6.36
Upper Mat South	-6.32	-7.54
Lower Mat South	-5.12	-6.34
Lower Mat North	-2.68	-1.46
Upper Mat North	-2.78	-1.56
Upper Mat Southeast	-6.12	-7.34
Upper Mat Northeast	-3.68	-2.46

Note: Negative sign indicates compressive pressure, positive sign indicates uplift pressure.

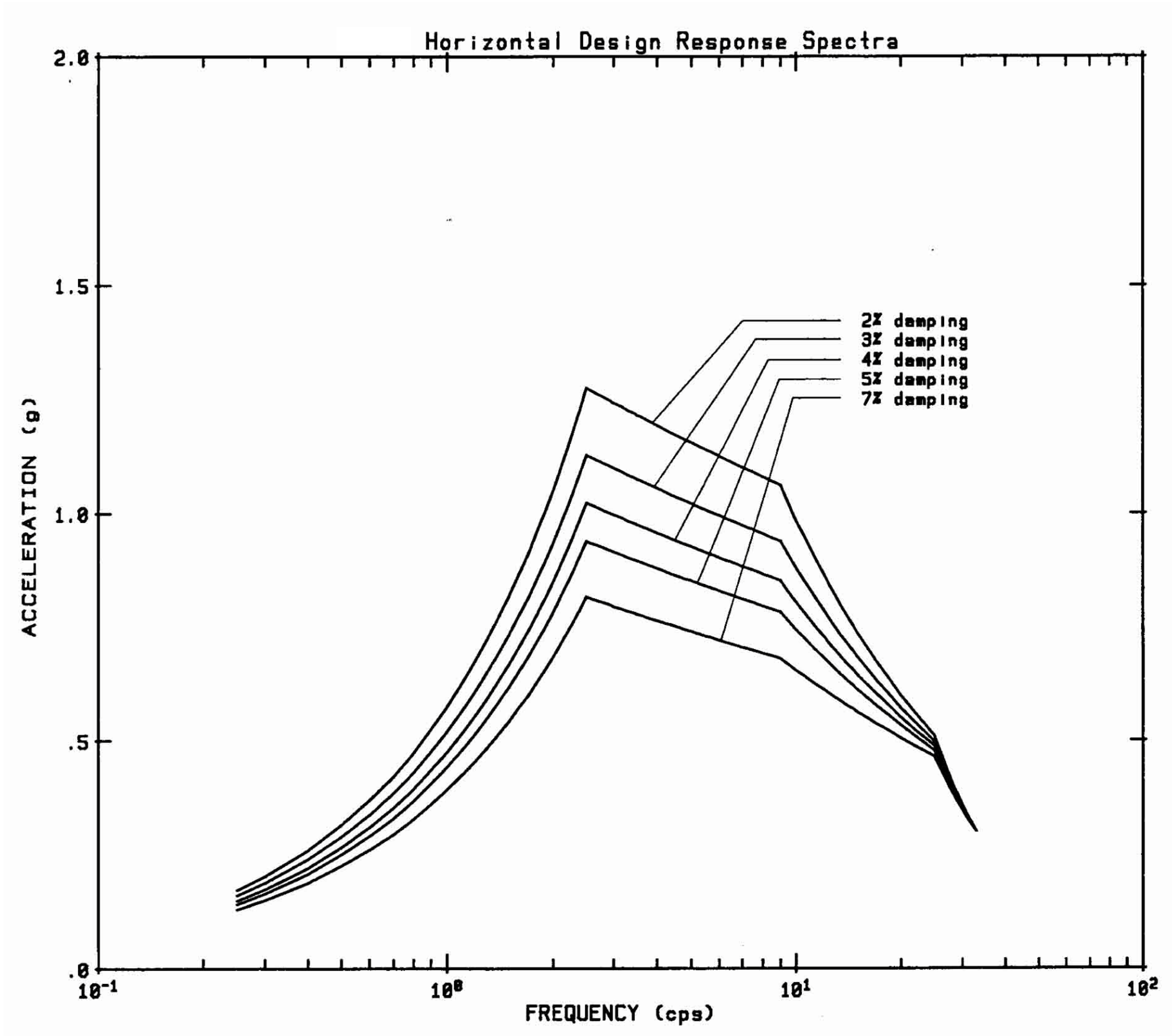


Figure 3.7.1-1
Horizontal Design Response Spectra
Safe Shutdown Earthquake

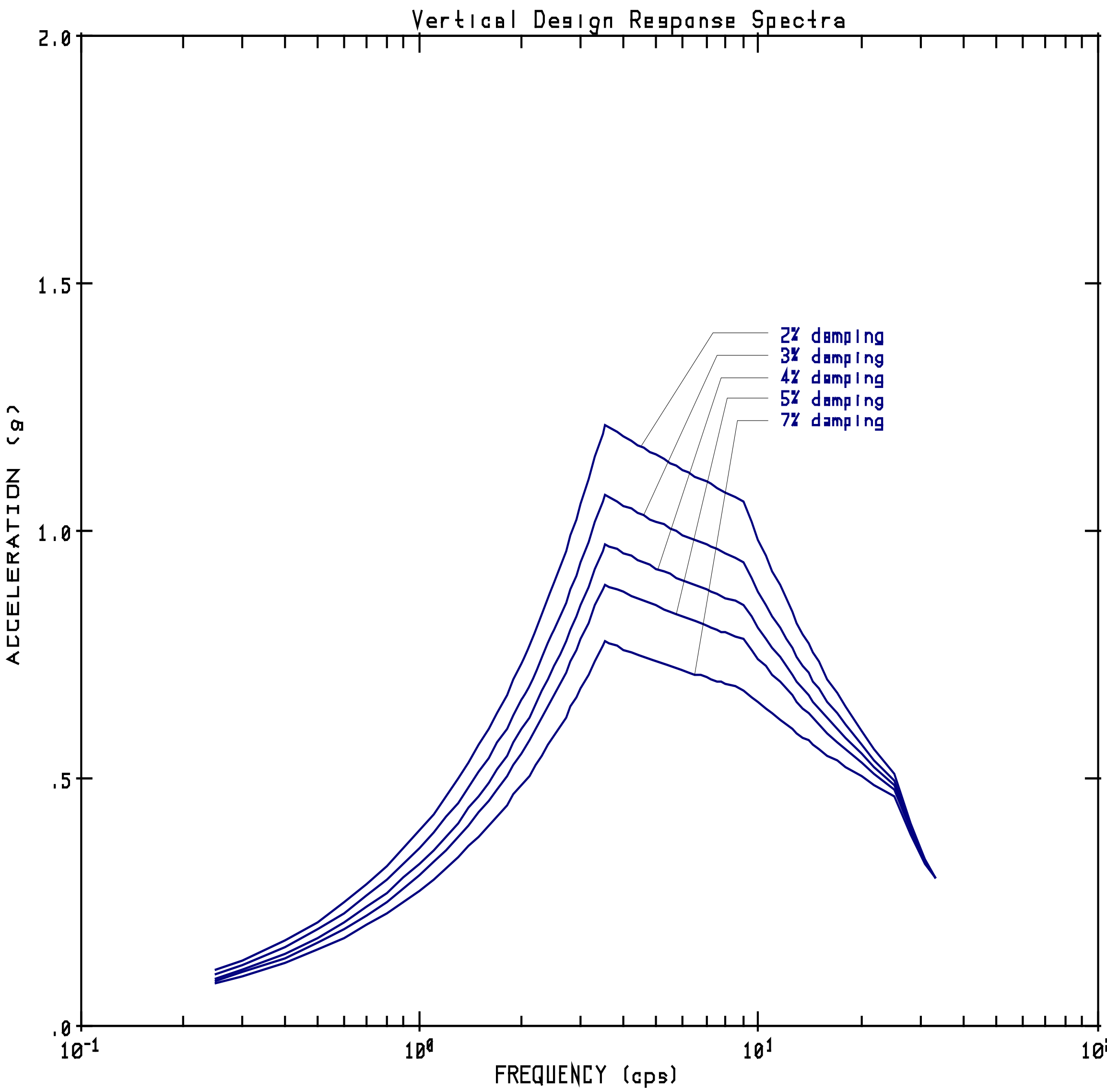


Figure 3.7.1-2
Vertical Design Response Spectra
Safe Shutdown Earthquake

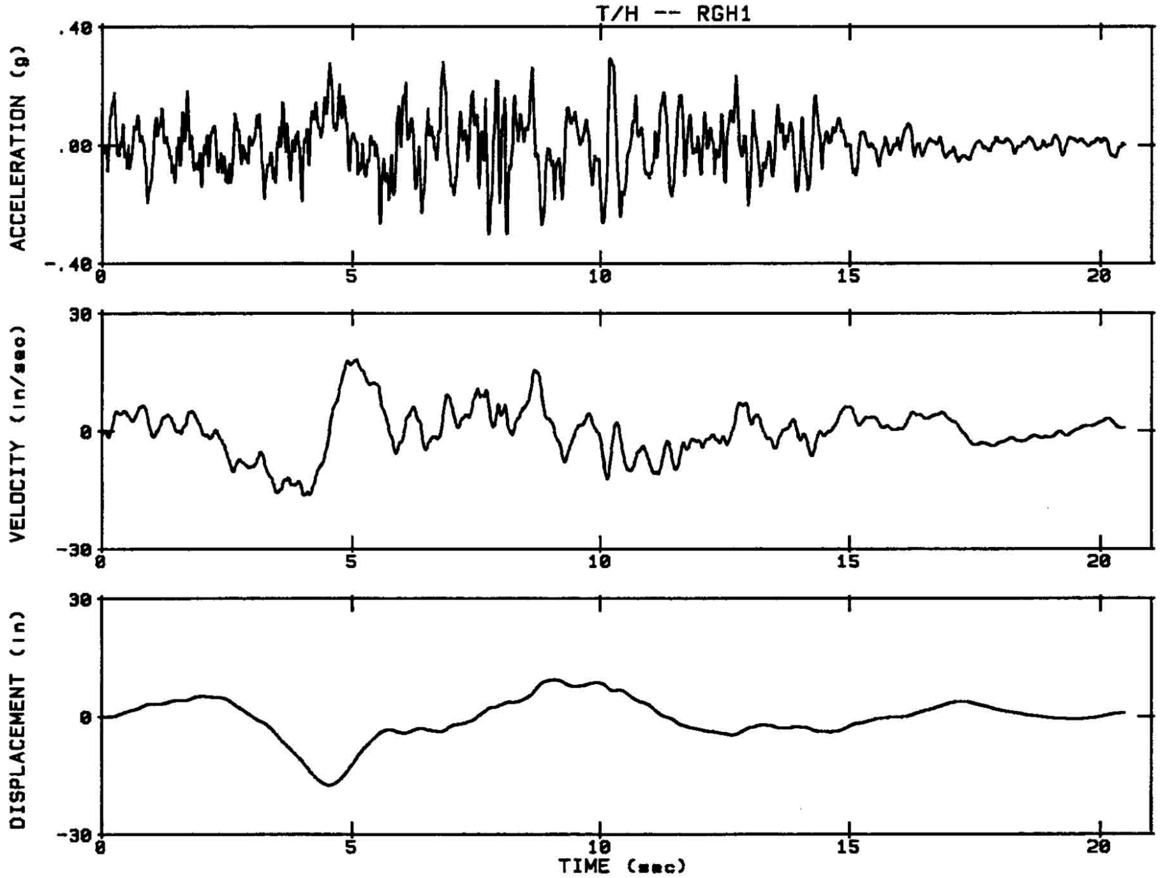


Figure 3.7.1-3
Design Horizontal Time History, "H1"
Acceleration, Velocity & Displacement Plots

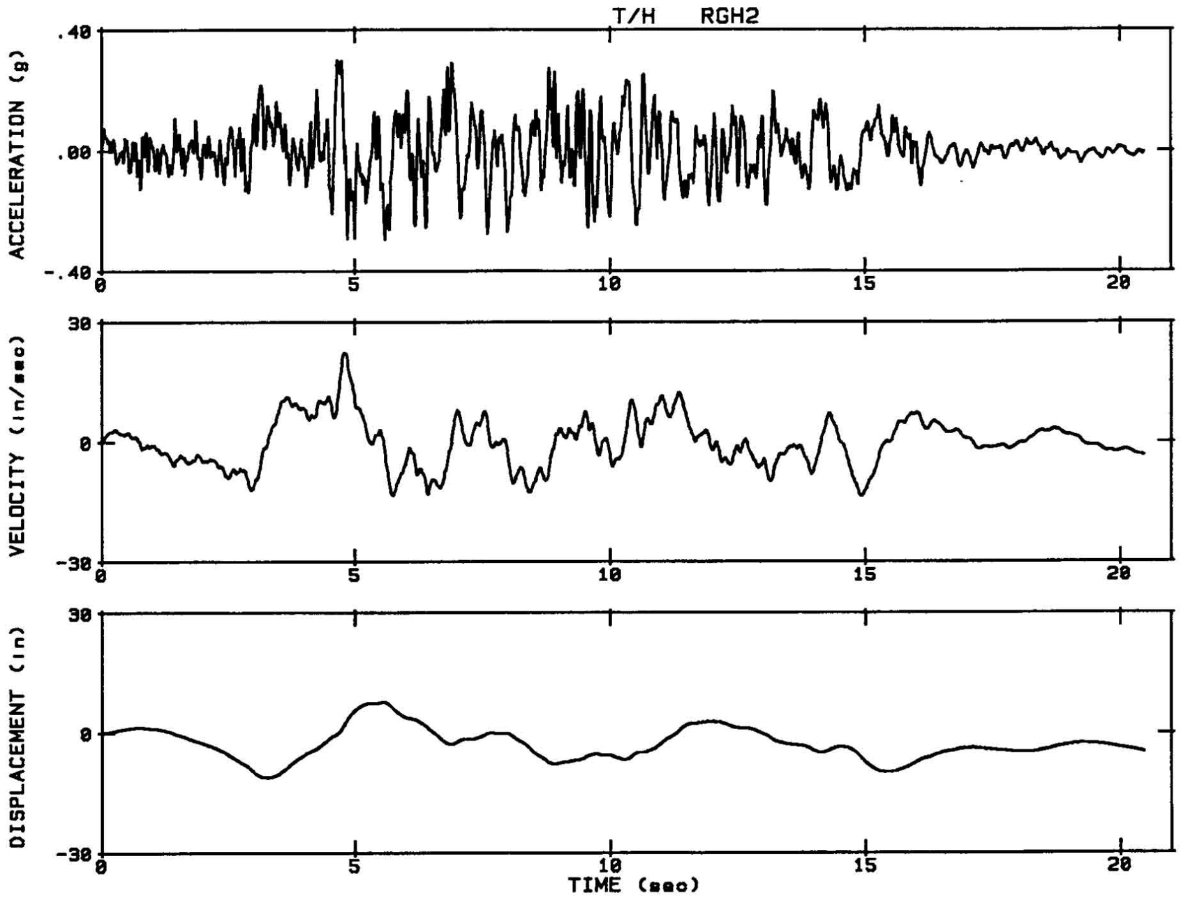


Figure 3.7.1-4
Design Horizontal Time History, "H2"
Acceleration, Velocity & Displacement Plots

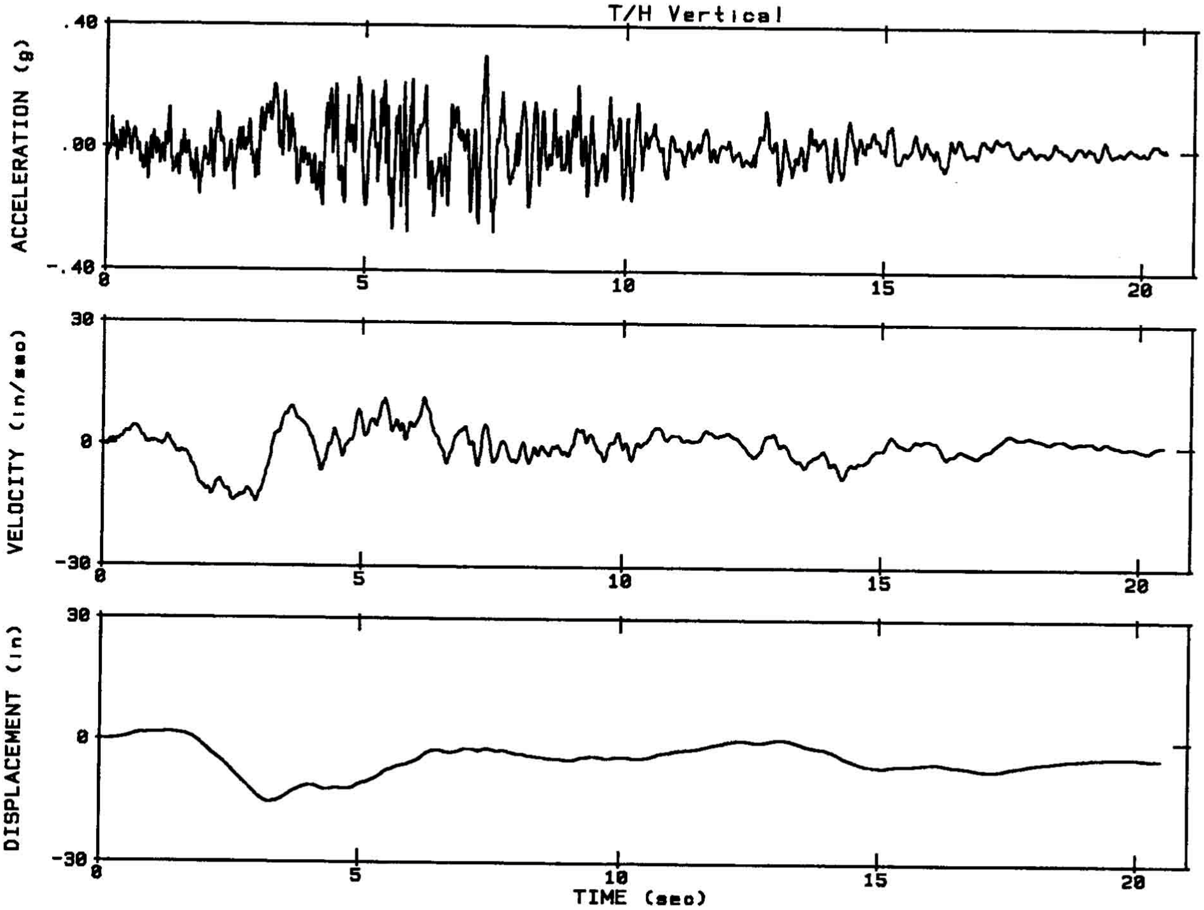


Figure 3.7.1-5
Design Vertical Time History
Acceleration, Velocity & Displacement Plots

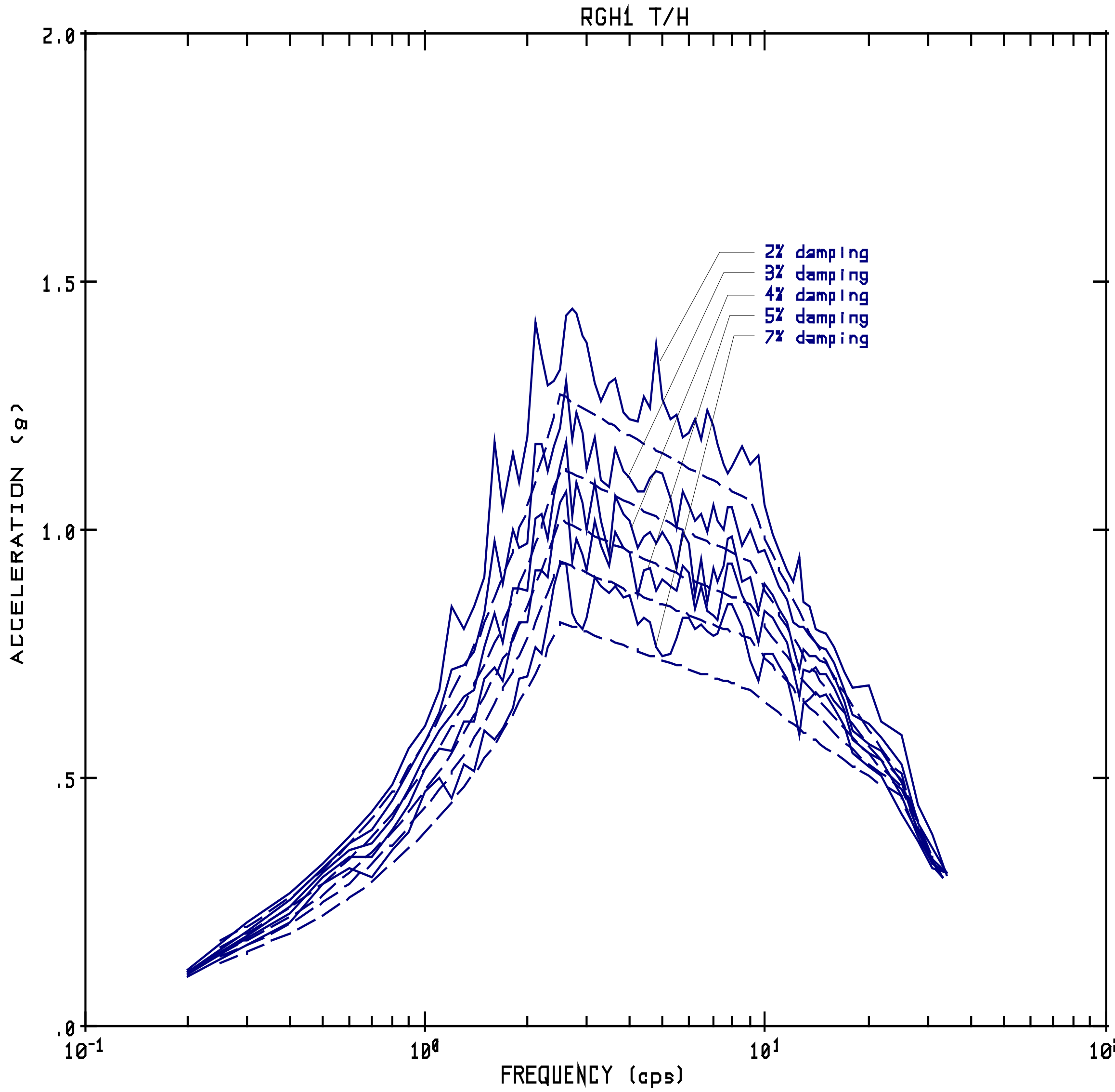


Figure 3.7.1-6
Acceleration Response Spectra of Design Horizontal Time History, "H1"

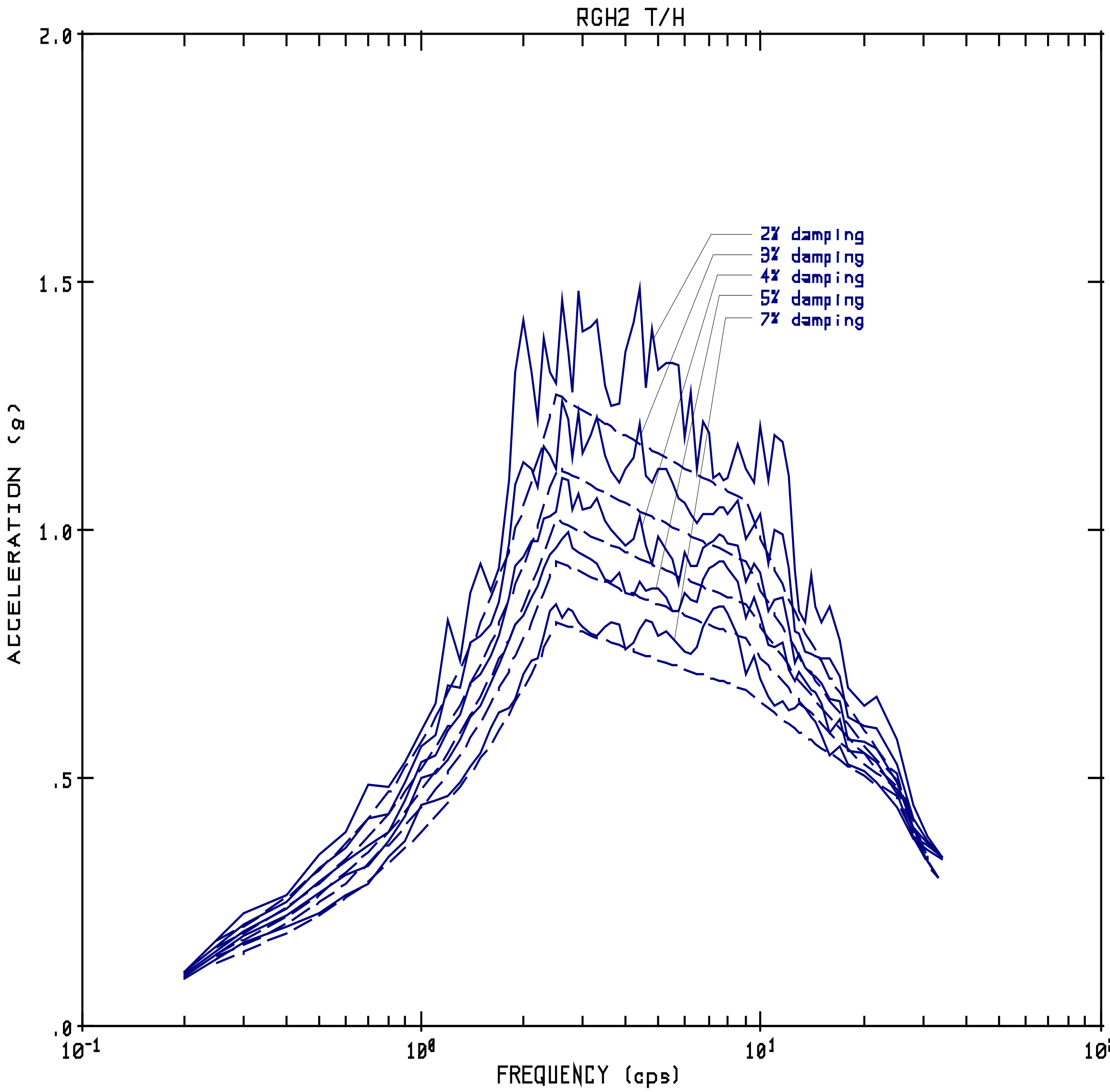


Figure 3.7.1-7
Acceleration Response Spectra of Design Horizontal Time History, "H2"

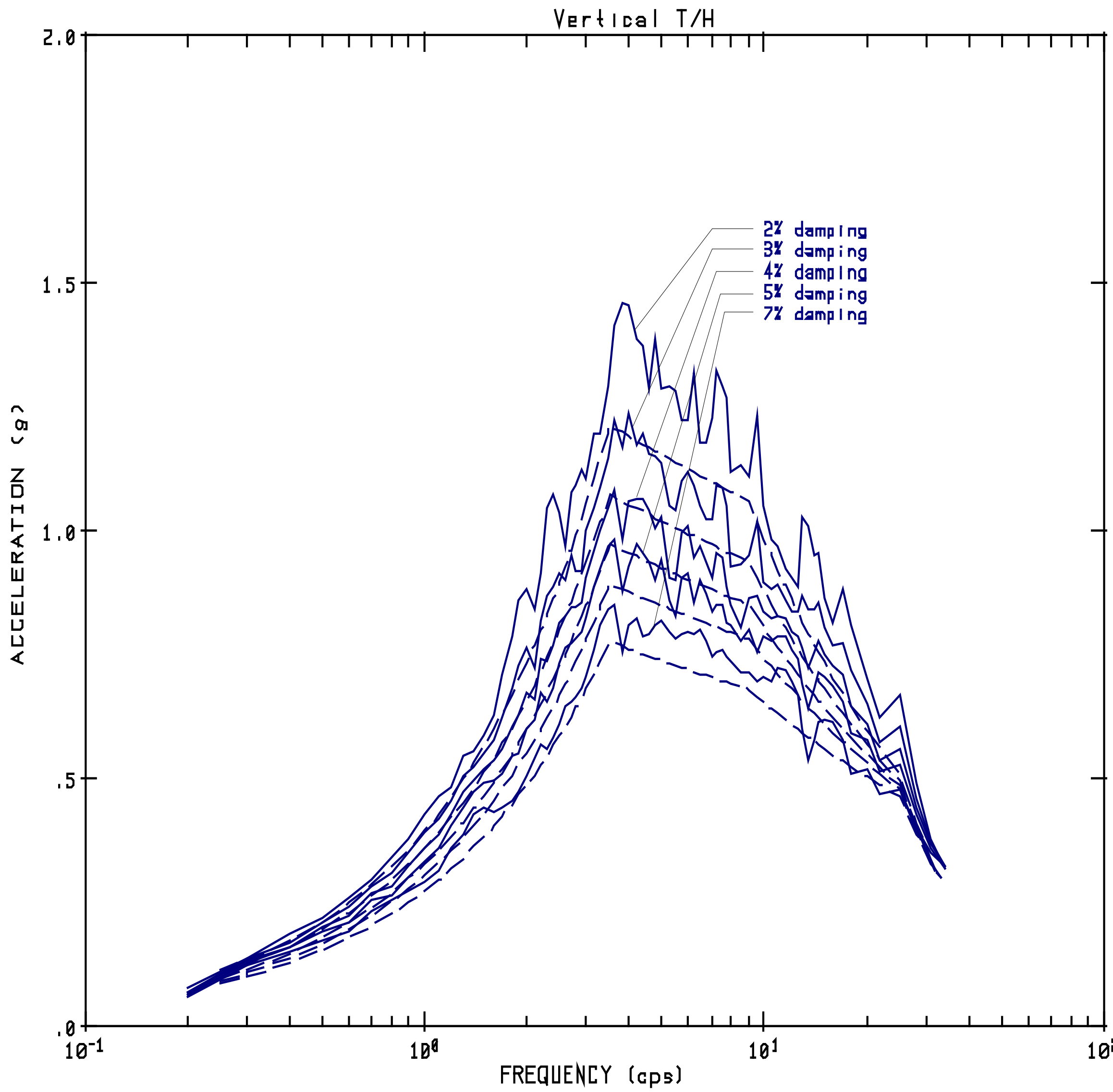


Figure 3.7.1-8
Acceleration Response Spectra of Design Vertical Time History

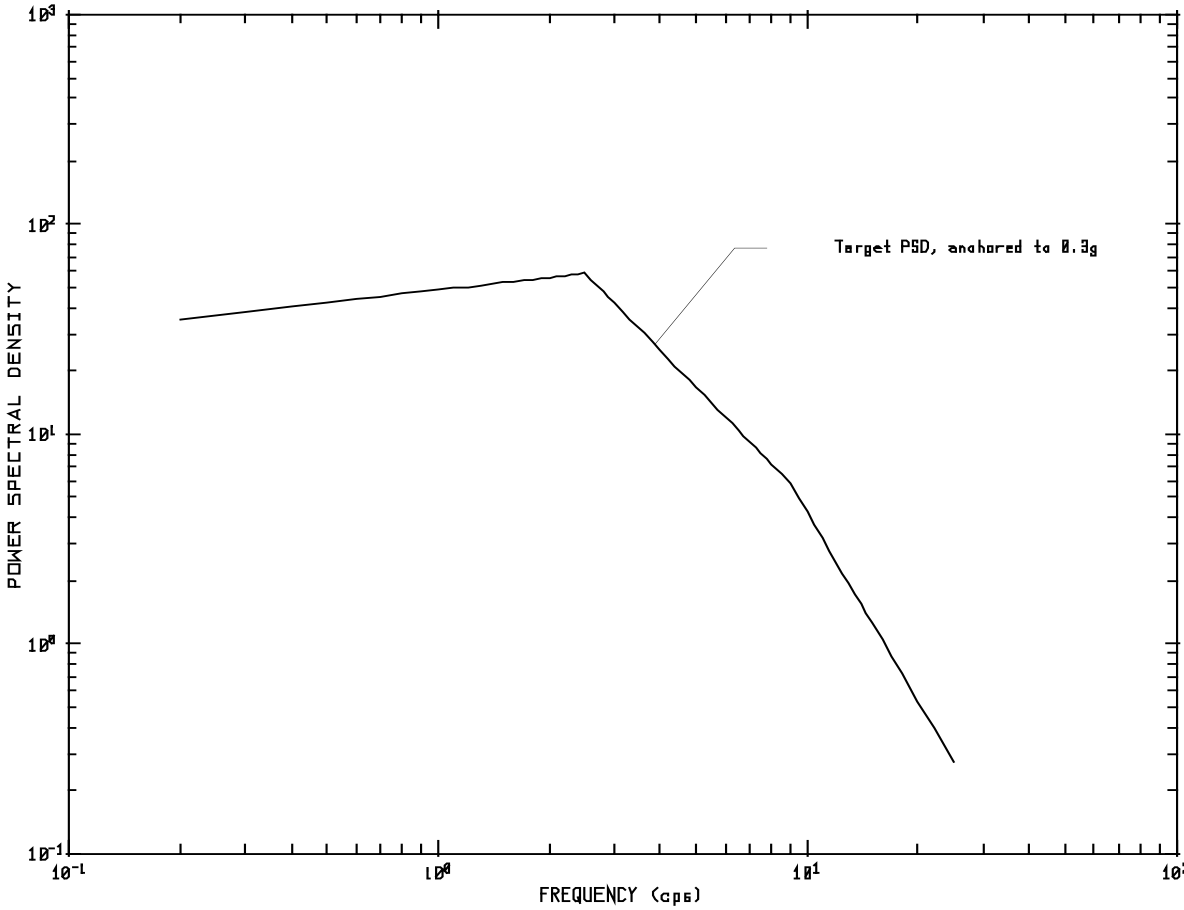


Figure 3.7.1-9
Minimum Power Spectral Density Curve
(Normalized to 0.3g)

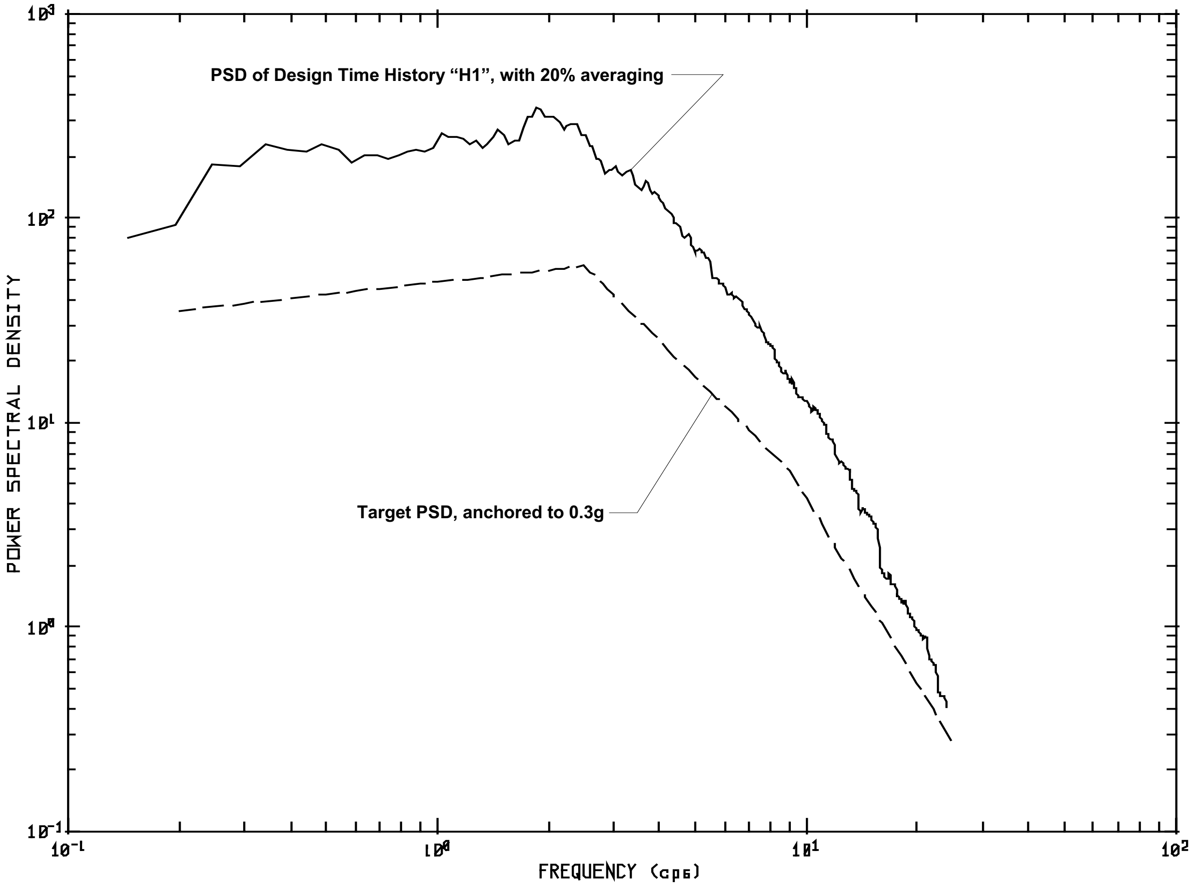


Figure 3.7.1-10
Power Spectral Density of
Design Horizontal Time History, "H1"

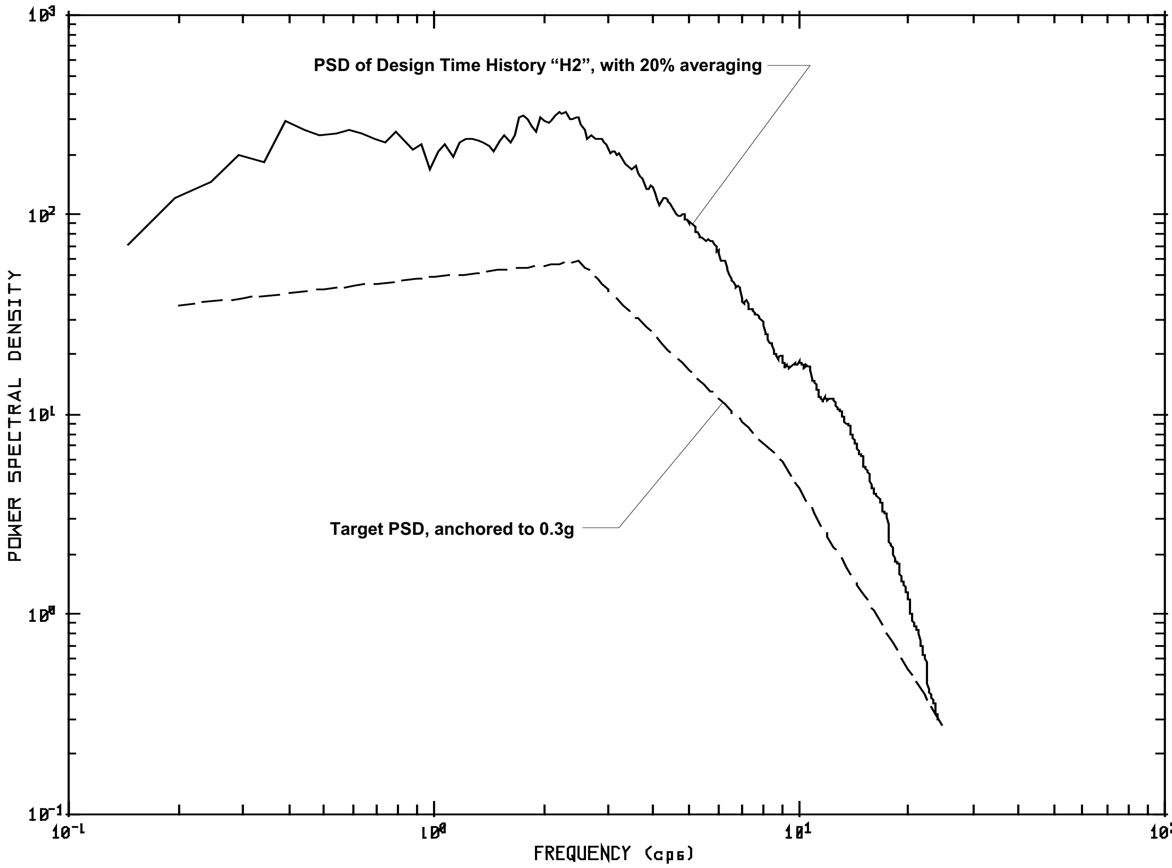


Figure 3.7.1-11
Power Spectral Density of
Design Horizontal Time History, "H2"

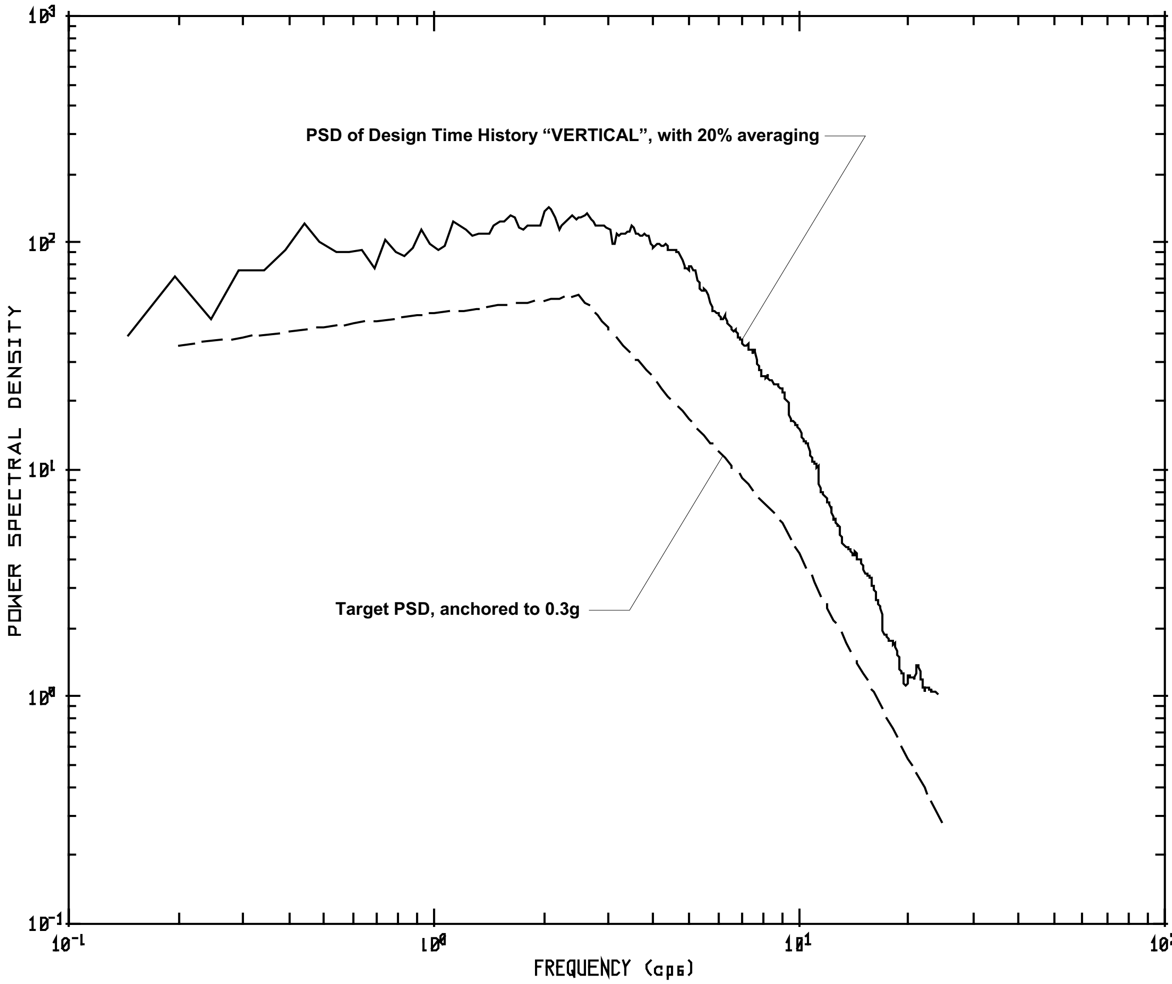


Figure 3.7.1-12
Power Spectral Density of
Design Vertical Time History

Figure 3.7.1-13 Not Used

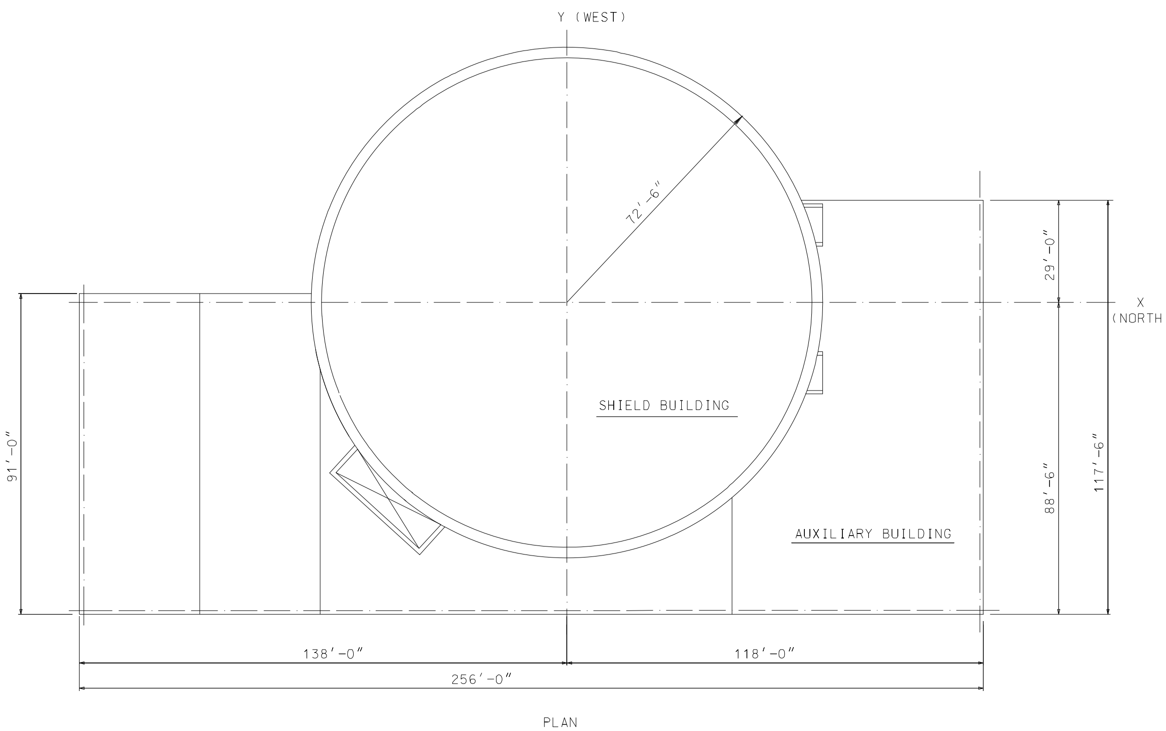
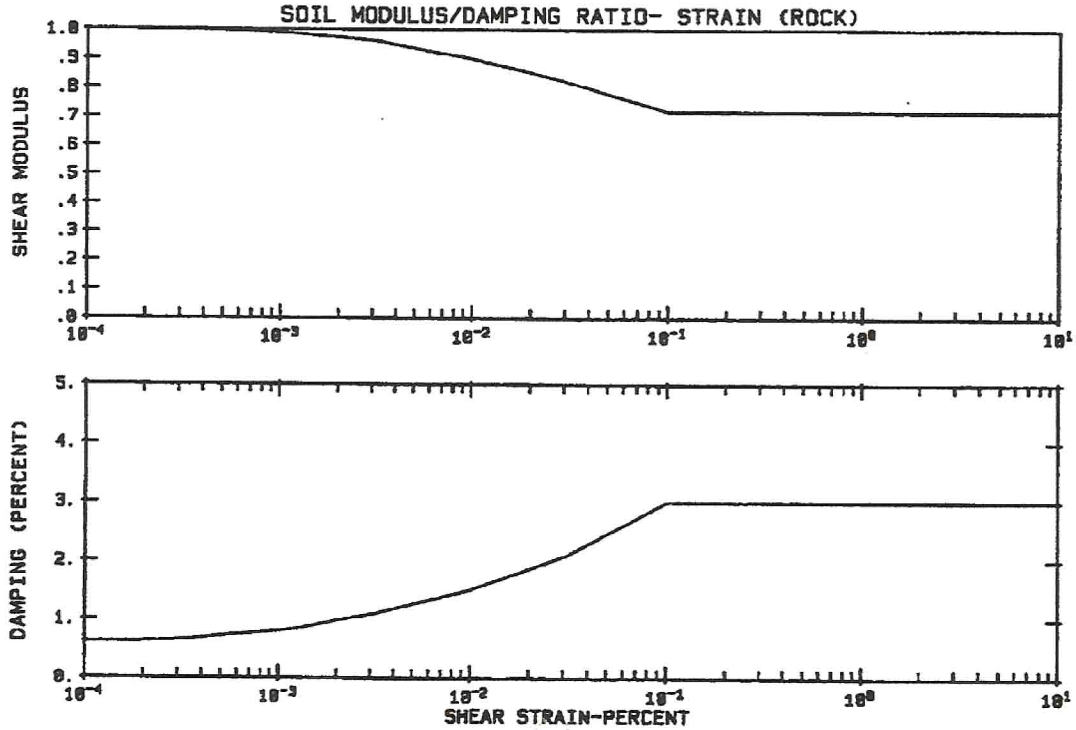


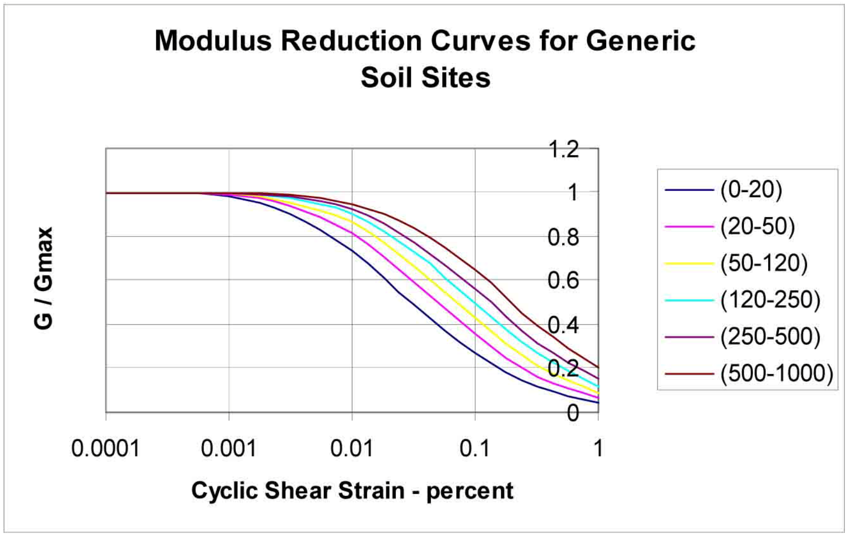
Figure 3.7.1-14
[*Nuclear Island Structures Dimensions*]*

*NRC Staff approval is required prior to implementing a change in this information.



Modulus Reduction Curves for Generic Soil Sites

Figure 3.7.1-15
Strain Dependent Properties of Rock Material (Ref. 37)



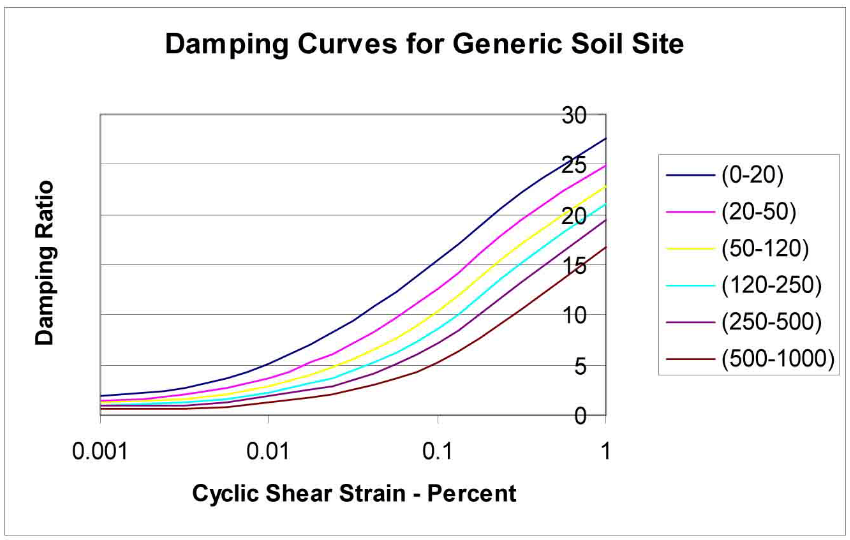
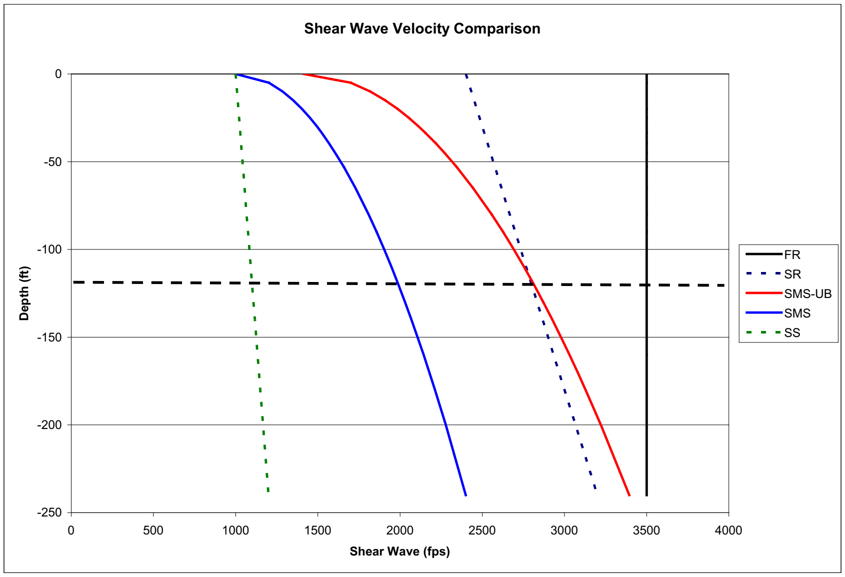
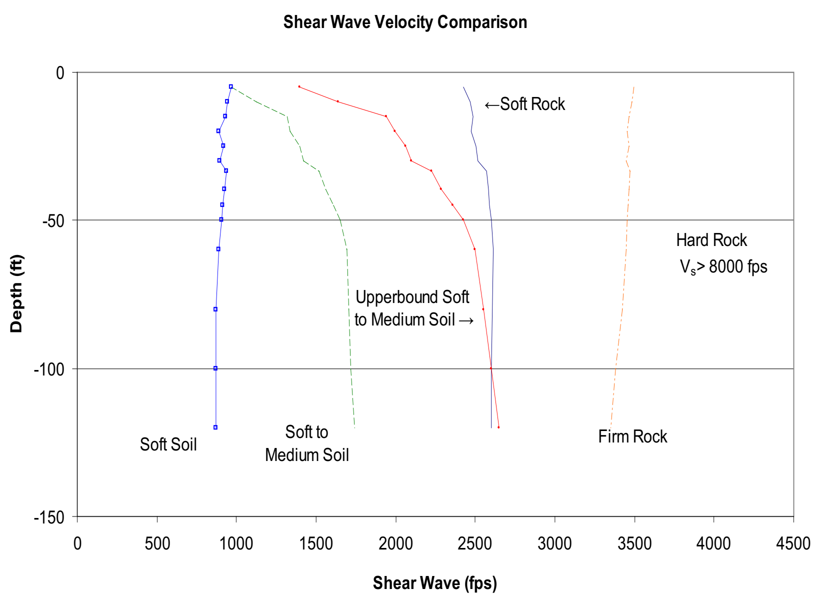


Figure 3.7.1-16
Strain Dependent Properties of Soil Material (Ref. 38)



Initial Properties



Strain-Iterated Shear Wave Velocity Profiles

Note: Fixed base analyses were performed for hard rock sites. These analyses are applicable for shear wave velocity greater than 8000 feet per second.

Figure 3.7.1-17
Generic Soil Profiles

Figures 3.7.2-1–3.7.2-11   Not Used

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.7.2-12 (Sheet 1 of 12)
[*Nuclear Island Key Structural Dimensions Plan at El. 66'-6"*]*

*NRC Staff approval is required prior to implementing a change in this information.

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.7.2-12 (Sheet 2 of 12)
[*Nuclear Island Key Structural Dimensions Plan at El. 82'-6"*]*

*NRC Staff approval is required prior to implementing a change in this information.

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.7.2-12 (Sheet 3 of 12)
[*Nuclear Island Key Structural Dimensions Plan at El. 100'-0"& 107'-2"*]*

*NRC Staff approval is required prior to implementing a change in this information.

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.7.2-12 (Sheet 4 of 12)
[*Nuclear Island Key Structural Dimensions Plan at El. 117'-6"*]*

*NRC Staff approval is required prior to implementing a change in this information.

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.7.2-12 (Sheet 5 of 12)
[*Nuclear Island Key Structural Dimensions Plan at El. 135'-3"*]*

*NRC Staff approval is required prior to implementing a change in this information.

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.7.2-12 (Sheet 6 of 12)
[*Nuclear Island Key Structural Dimensions Plan at El. 153'-0" & 160'-6"*]*

*NRC Staff approval is required prior to implementing a change in this information.

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.7.2-12 (Sheet 7 of 12)
[*Nuclear Island Key Structural Dimensions Plan at El. 160'-6", 180'-0", & 329'-0"*]*

*NRC Staff approval is required prior to implementing a change in this information.

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.7.2-12 (Sheet 8 of 12)
[*Nuclear Island Key Structural Dimensions Section A - A*]*

*NRC Staff approval is required prior to implementing a change in this information.

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.7.2-12 (Sheet 9 of 12)
[*Nuclear Island Key Structural Dimensions Section B - B*]*

*NRC Staff approval is required prior to implementing a change in this information.

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.7.2-12 (Sheet 10 of 12)
[*Nuclear Island Key Structural Dimensions Sections C - C and H - H*]*

*NRC Staff approval is required prior to implementing a change in this information.

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.7.2-12 (Sheet 11 of 12)
[*Nuclear Island Key Structural Dimensions Section G - G*]*

*NRC Staff approval is required prior to implementing a change in this information.

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.7.2-12 (Sheet 12 of 12)
[*Nuclear Island Key Structural Dimensions Section J - J*]*

*NRC Staff approval is required prior to implementing a change in this information.

Figure 3.7.2-13 Not Used

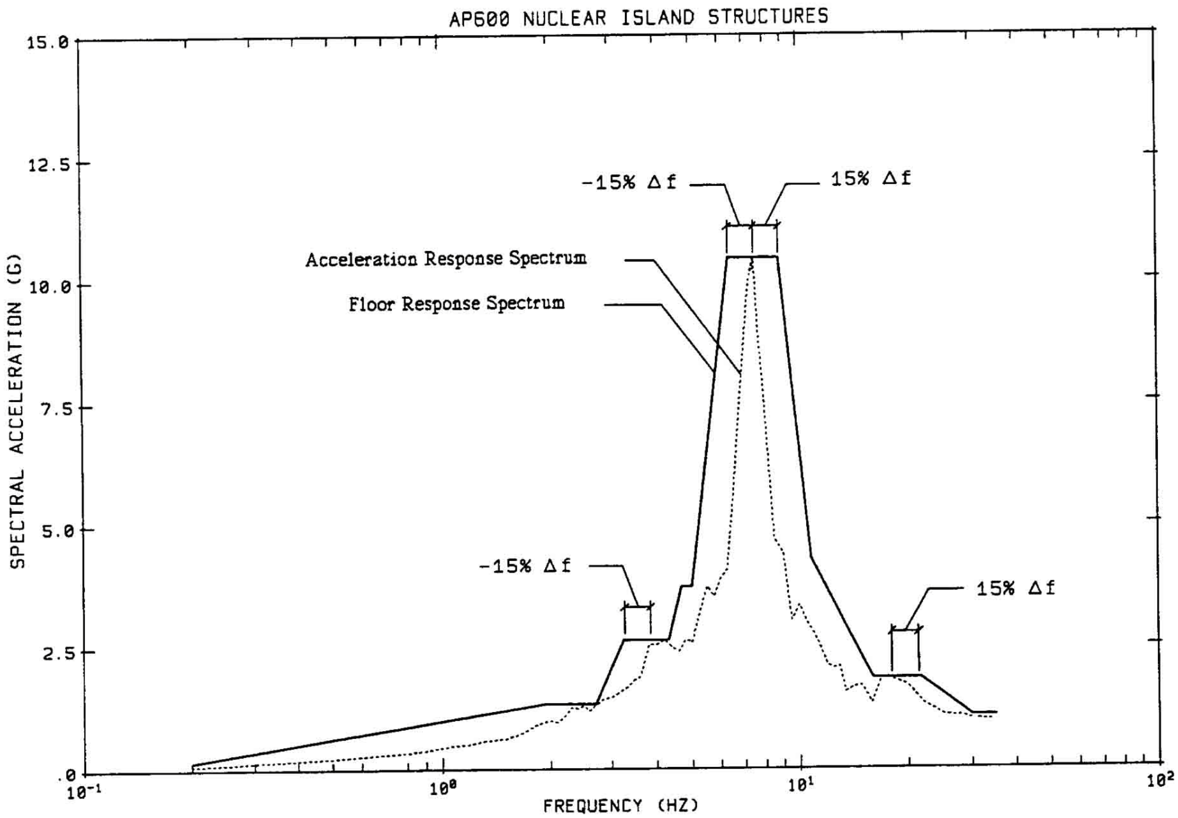


Figure 3.7.2-14
Typical Design Floor Response Spectrum

Figures 3.7.2-15–3.7.2-18 Not Used

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.7.2-19 (Sheet 1 of 10)
Annex Building Key Structural Dimensions
Plan at Elevation 100′-0″

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.7.2-19 (Sheet 2 of 10)
Annex Building Key Structural Dimensions
Plan at Elevation 107′-2″ and 117′-6″

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.7.2-19 (Sheet 3 of 10)
Annex Building Key Structural Dimensions
Plan at Elevation 135′-3″

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.7.2-19 (Sheet 4 of 10)
Annex Building Key Structural Dimensions
Plan at Elevation 158′-0″ and 146′-3″

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.7.2-19 (Sheet 5 of 10)
Annex Building Key Structural Dimensions
Roof Plan at Elevation 154′-0″ and 181′-11 3/4″

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.7.2-19 (Sheet 6 of 10)
Annex Building Key Structural Dimensions
Section A - A

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.7.2-19 (Sheet 7 of 10)
Annex Building Key Structural Dimensions
Section B - B

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.7.2-19 (Sheet 8 of 10)
Annex Building Key Structural Dimensions
Section C - C

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.7.2-19 (Sheet 9 of 10)
Annex Building Key Structural Dimensions
Sections D - D, E - E, & F - F

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.7.2-19 (Sheet 10 of 10)
Annex Building Key Structural Dimensions
Sections G - G, H - H, & J - J

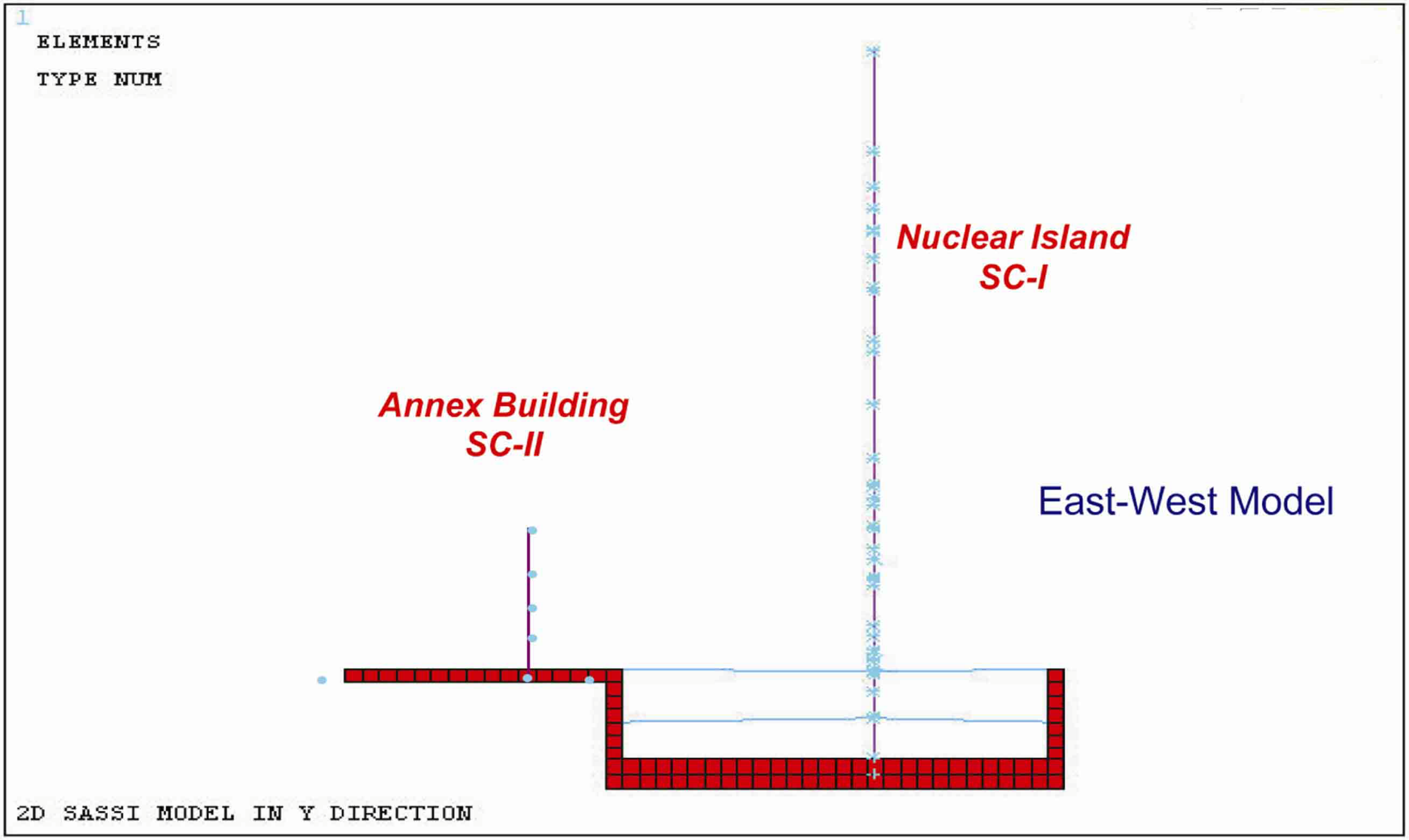


Figure 3.7.2-20
East-West 2D SASSI Model with Adjacent Buildings

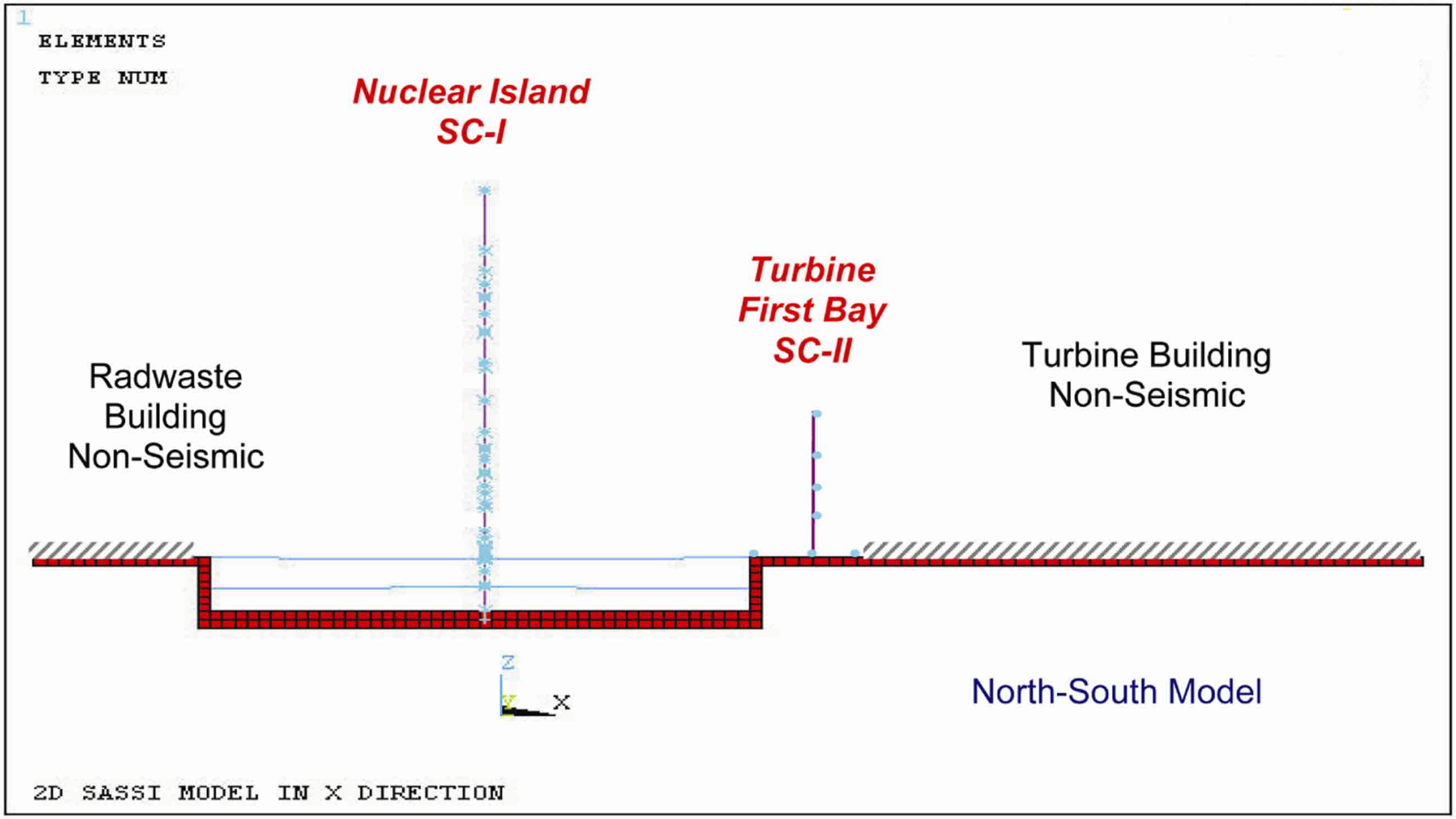


Figure 3.7.2-21
2D North-South SASSI Model with Adjacent Buildings

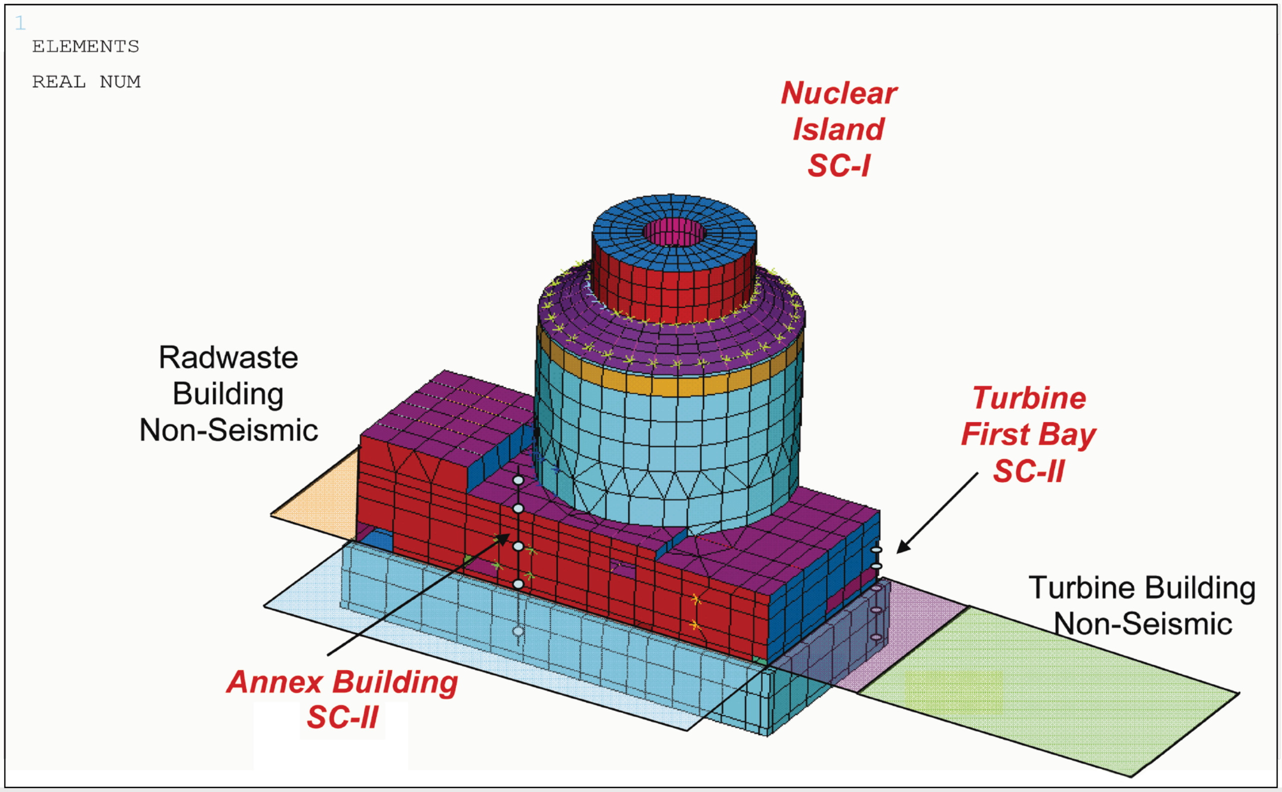


Figure 3.7.2-22
3D SASSI Model with Adjacent Buildings

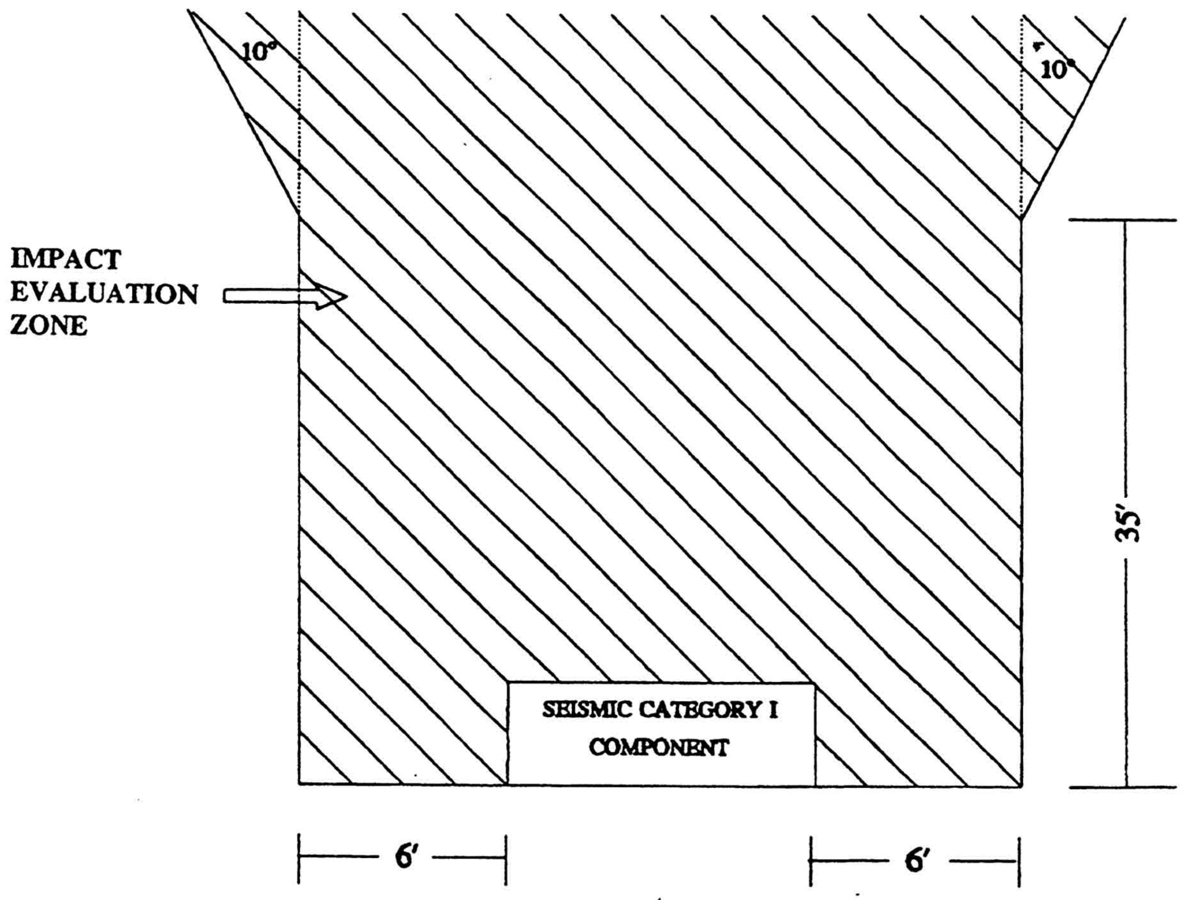


Figure 3.7.3-1
Impact Evaluation Zone

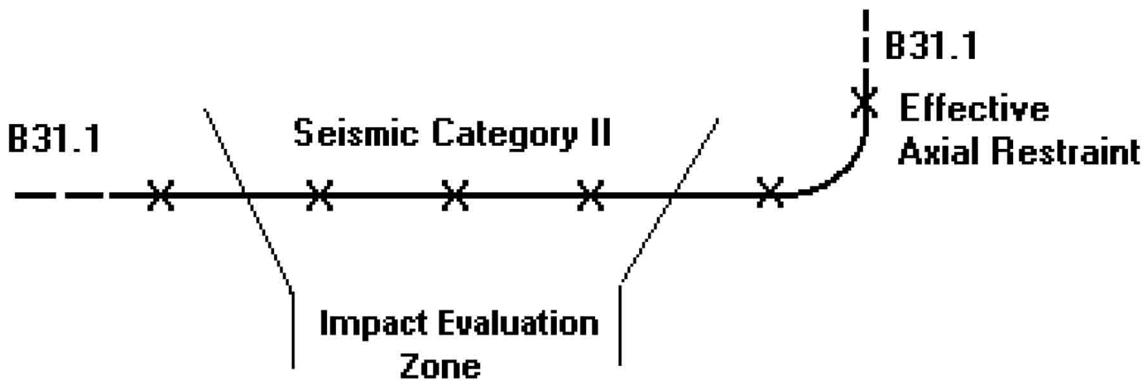


Figure 3.7.3-2
Impact Evaluation Zone and Seismic Supported Piping

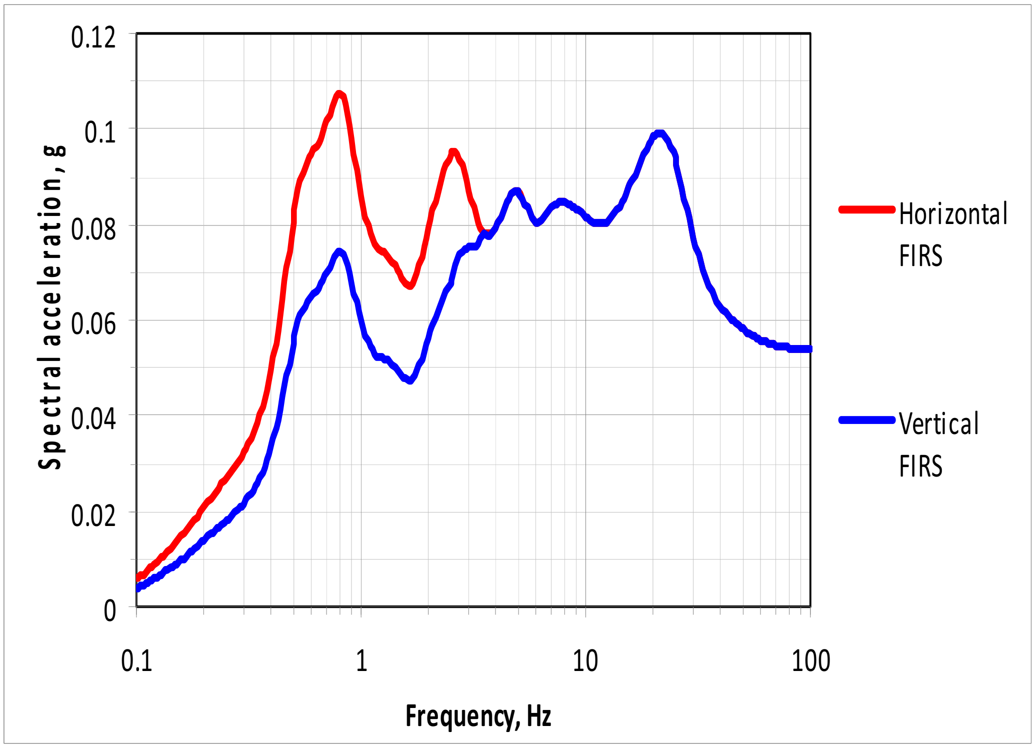
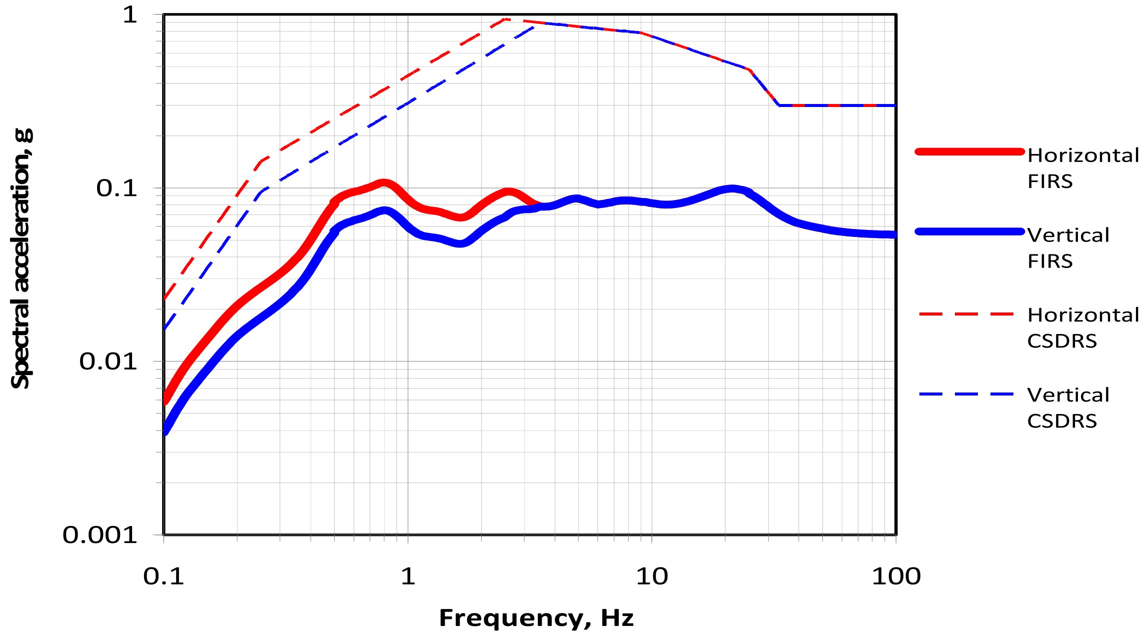
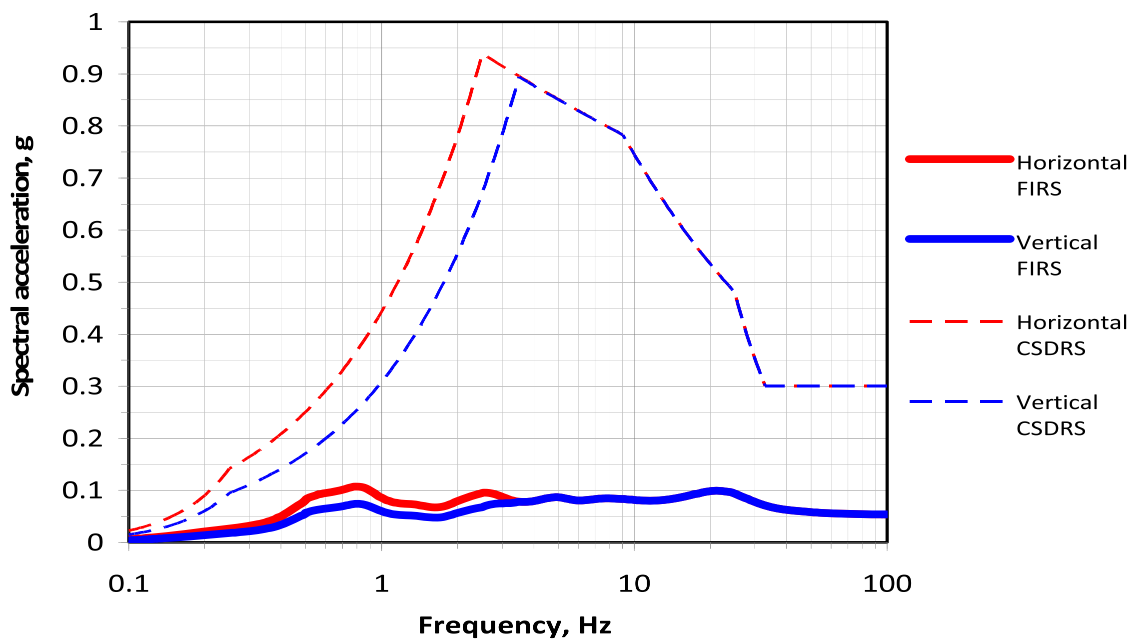


Figure 3.7-201 Recommended Horizontal and Vertical FIRS (Elevation –16 foot Horizon at Bottom of Nuclear Island Foundation)



(Log-Log Format)



(Log-Linear Format)

Figure 3.7-202 Comparison of Turkey Point Horizontal and Vertical FIRS with AP1000 Horizontal and Vertical CSDRS

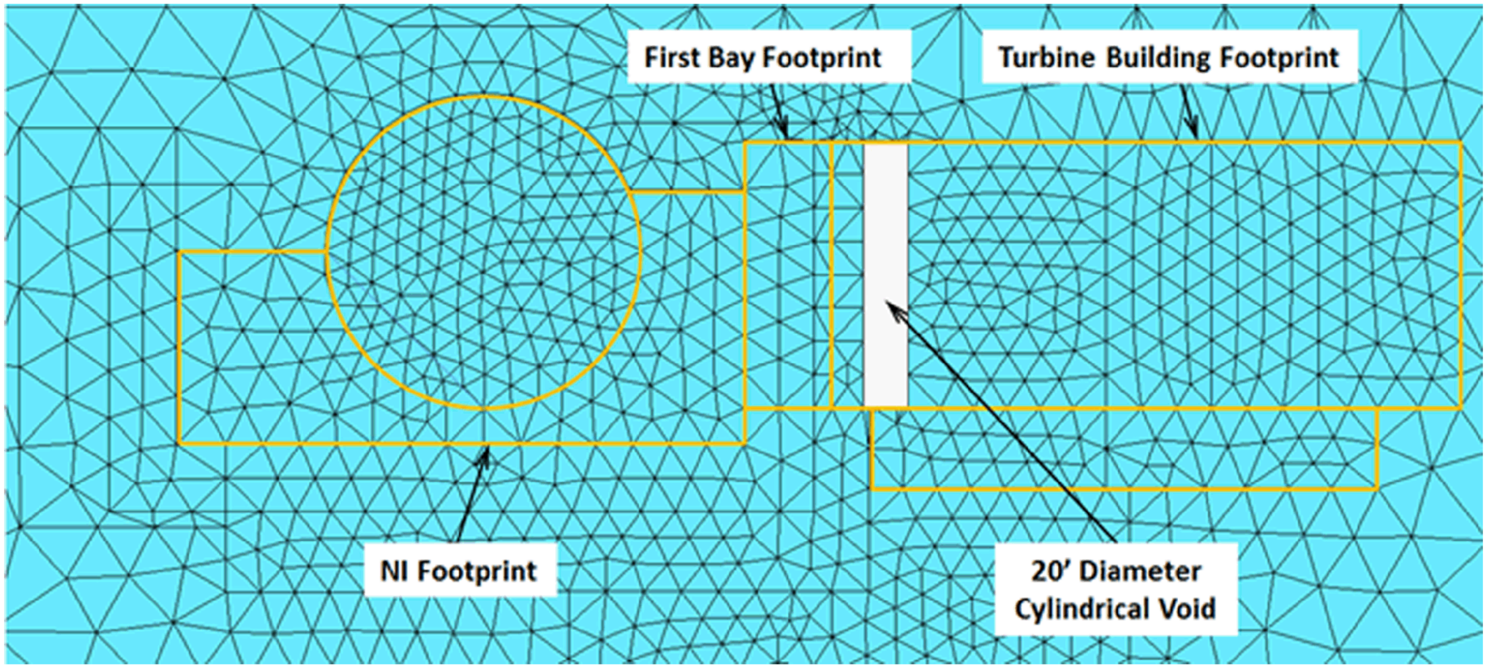


Figure 3.7-203 PLAXIS 3D Sensitivity Model with 20-foot Diameter Cylindrical Void

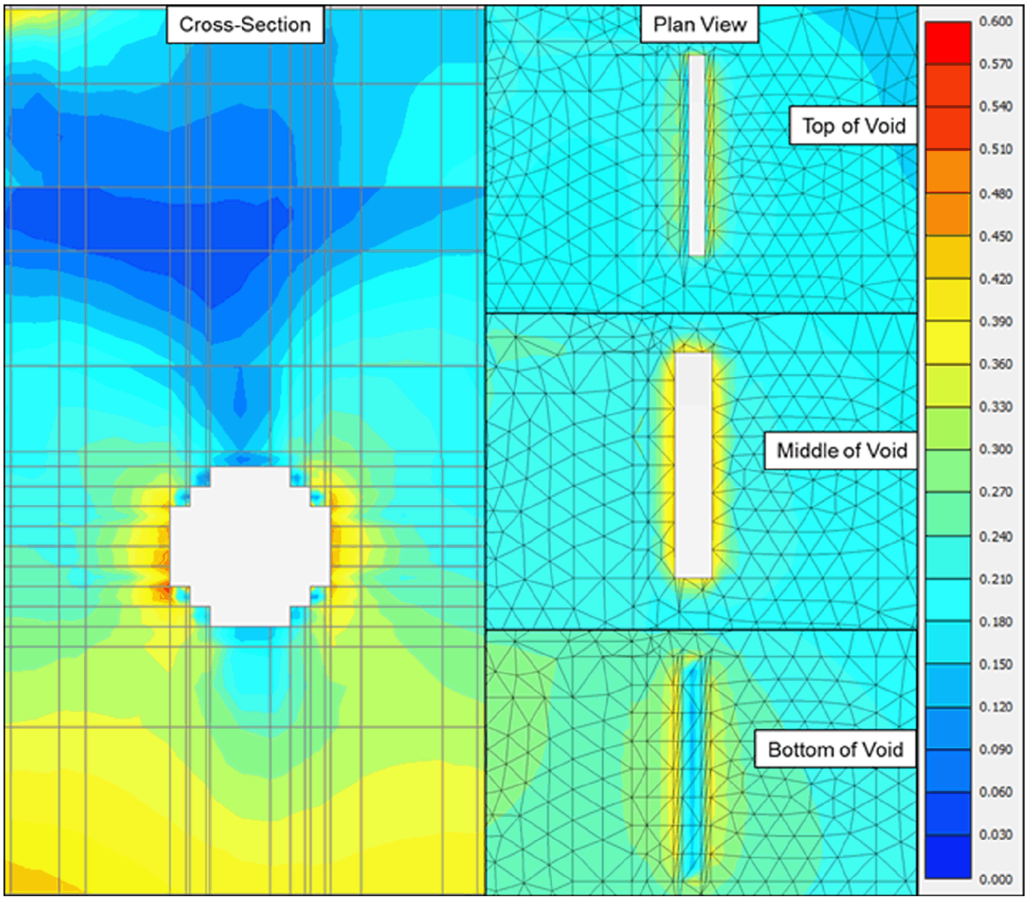


Figure 3.7-204 Relative Shear Stresses (Static and Pseudo-Dynamic Loading, Multiplier of 1 as defined in Table 3.7-202)

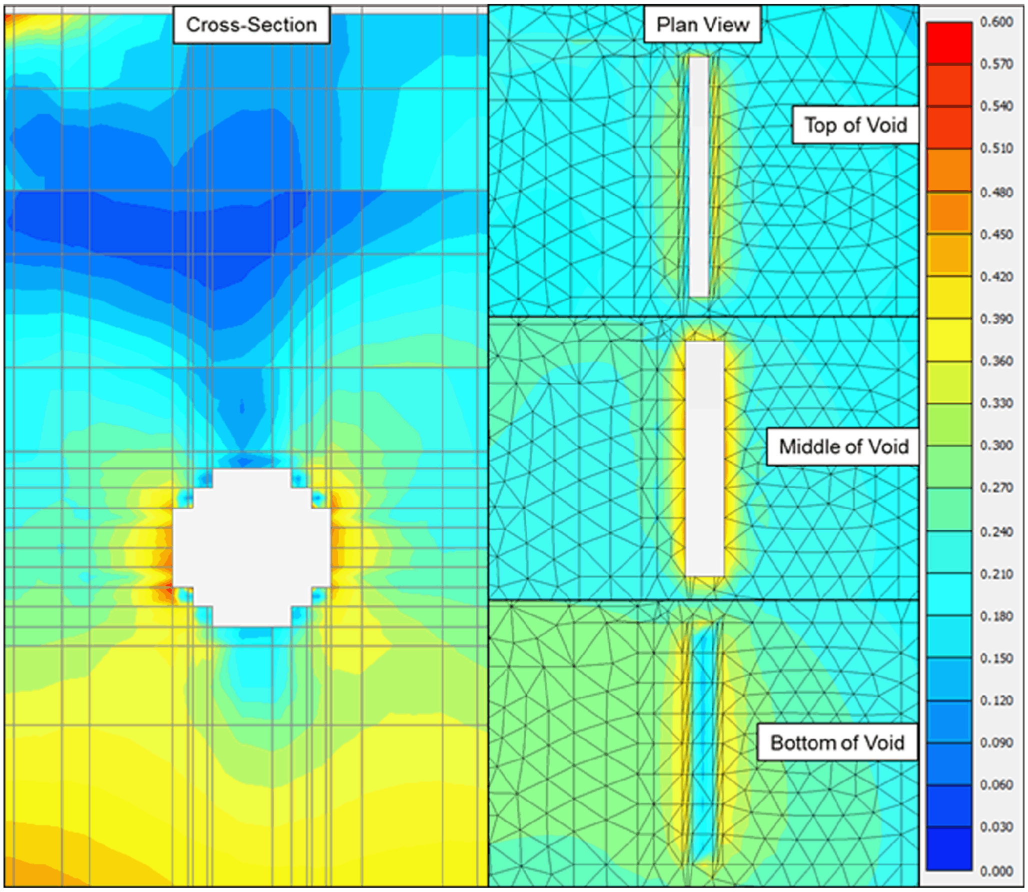


Figure 3.7-205 Relative Shear Stresses (Static and Pseudo-Dynamic Loading, Multiplier of 2 as defined in Table 3.7-202)